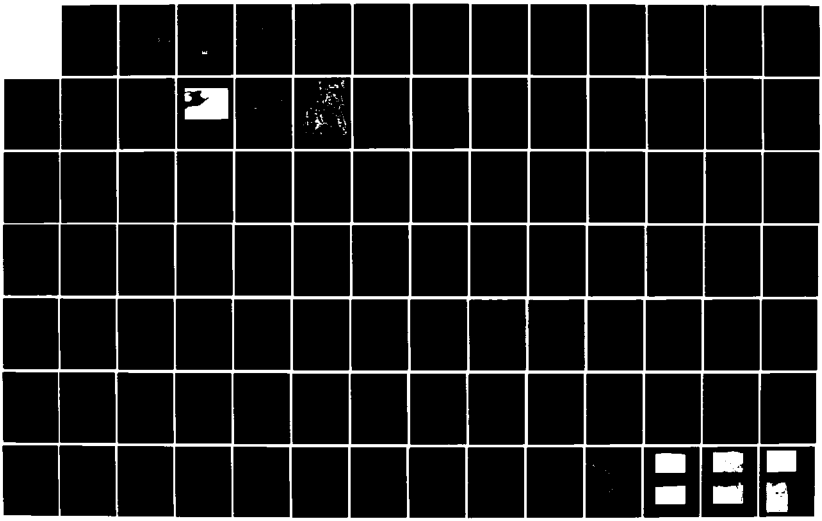
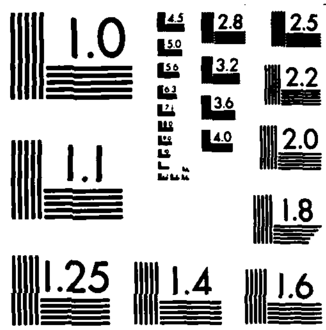
AD-A156 023 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS SUGAR HILL DAM (VT 00..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV FEB 80 1/2

UNCLASSIFIED F/G 13/13 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

ACTION
PHOTOGRAPH THIS SHEET

II	1
LEVEL	INVENTORY

AD-A156 023

DTIC ACCESSION NUMBER

SUGAR Hill DAM VT 00176

DOCUMENT IDENTIFICATION

This document has been approved for public release and sale; its distribution is unlimited.

DISTRIBUTION STATEMENT

ACCESSION FOR	
NTIS GRA&I	☒
DTIC TAB	☐
UNANNOUNCED	☐
JUSTIFICATION	
BY	
DISTRIBUTION /	
AVAILABILITY CODES	

DIST	AVAIL AND/OR SPECIAL	
A-1		

DISTRIBUTION STAMP

DTIC
SELECTED
JUL 0 5 1985
E

DATE ACCESSIONED

DATE RETURNED

85 7 03 089

DATE RECEIVED IN DTIC

REGISTERED OR CERTIFIED NO.

PHOTOGRAPH THIS SHEET AND RETURN TO DTIC-DDAC

RICHELIEU RIVER BASIN
TOWN OF GOSHEN
ADDISON COUNTY, VERMONT

SUGAR HILL DAM
VT 00176

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MA 02154

FEBRUARY 1980

REPRODUCED AT GOVERNMENT EXPENSE

DISCLAIMER NOTICE

THIS DOCUMENT IS BEST QUALITY PRACTICABLE. THE COPY FURNISHED TO DTIC CONTAINED A SIGNIFICANT NUMBER OF PAGES WHICH DO NOT REPRODUCE LEGIBLY.

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER VT 00179	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Sugar Hill Dam NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS		5. TYPE OF REPORT & PERIOD COVERED INSPECTION REPORT 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION, NEDED 424 TRAPELO ROAD, WALTHAM, MA. 02254		12. REPORT DATE February 1980 13. NUMBER OF PAGES 130
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) UNCLASSIFIED 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

APPROVAL FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Richelieu River Basin
Town of Goshen
Addison County VT.
Sucker Brook

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam is an earth embankment about 855 ft long and 61 ft. high. THe dam is in fair condition. The spillwau approach and discharge channels are becoming overgrown with brush and small trees and are in need of maintenance. It is intermediate in size with a high hazard potential. There are recommendations which must be undertaken by the owener.

RICHELIEU RIVER BASIN
TOWN OF GOSHEN
ADDISON COUNTY, VERMONT

SUGAR HILL DAM
VT 00176

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

GORDON E. AINSWORTH & ASSOCIATES, INC.
Engineers, Surveyors and Planners
20 SUGARLOAF ST. SOUTH DEERFIELD, MASS. 01373

PROJECT NO. 21. 06. 79106

LETTER OF TRANSMITTAL

FROM THE CORPS OF ENGINEERS TO THE STATE

TO BE SUPPLIED BY THE CORPS OF ENGINEERS

NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

Identification No.:	VT 00176
Name of Dam:	Sugar Hill Dam
Town:	Goshen
County and State:	Addison County, Vermont
Stream:	Sucker Brook
Date of Inspection:	7 November 1979

BRIEF ASSESSMENT

1. Project Description

Sugar Hill Dam is an earth embankment about 855 feet long by 61 feet high. Included in the length of the dam is a 150-foot long spillway in the right abutment. Both the upstream and downstream slopes of the embankment are about 2H:1V. Top width is about 16 feet.

Normal pool elevation is maintained somewhat below the spillway crest by a regulating outlet conduit under the dam controlled at its downstream end. The spillway crest is about 3.5 feet below the lowest point on top of the dam. The only spillway is the spillway in the right abutment, which consists of a short approach section, a free overflow concrete control weir, and a discharge channel back to Sucker Brook.

2. Significant Findings and Assessment

The dam is in FAIR condition. Significant problems include erosion around the outlet structure and at the upstream left abutment contact line, a thin crack-like line transversely crossing the downstream portion of the crest, brush and decaying stumps on the downstream slope, structural deterioration of portions of the outlet structure, and deterioration of the left training wall of the spillway. Also, the spillway approach and discharge channels are becoming overgrown with brush and small trees and are in need of maintenance.

3. Hydraulic and Hydrologic Findings

The spillway is INADEQUATE to pass the test flood without overtopping the dam. In accordance with recommended guidelines of the Corps of Engineers, the dam is classified as INTERMEDIATE

in size and as having a HIGH hazard potential. Accordingly, a TEST FLOOD equal to FULL PMF (probable maximum flood) is required. The test flood overtops the dam by a maximum of about 0.6 of a foot with duration of overtopping of about 3.7 hours. Peak inflow for the test flood is 4300 cfs. Peak outflow is reduced very little by reservoir routing and is 4200 cfs. Total project discharge capacity at the top of the dam is due only to the spillway (outlet works assumed closed) and is equal to 3030 cfs, or 72% of the test flood peak outflow.

A flood equal to HALF PMF does not overtop the dam, but results in a minimum freeboard of about 0.8 of a foot.

4. <u>Recommended Action</u>

WITHIN ONE YEAR after their receipt of this Phase I Inspection Report, the Owner should implement the following recommendations:

a. All brush and stumps should be removed from upstream and downstream slopes, and the spillway approach and discharge channels should be cleared of brush, trees, and logs and maintained, all as described in Section 7 of this report.

b. A registered engineer qualified in the design of earth dams should be engaged to do such work as inspect the dam after it has been cleared of brush, thoroughly inspect the inside of the outlet conduit and outlet structure, determine the best method of repair for the outlet structure, and perform a detailed hydraulic and hydrologic study to better assess spillway capacity. Contingent on the results of the detailed hydraulic and hydrologic study, determine what repairs should be made to the left training wall of the spillway. Necessary spillway repairs and/or enlargement should be carried out.

Additional recommendations and remedial measures that should be implemented by the Owner WITHIN ONE YEAR after receipt of this Phase I Inspection Report are described in Section 7.

GORDON E. AINSWORTH & ASSOCIATES, INC.

Kenneth J. Male, P.E.

This Phase I Inspection Report on Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

THIS SHEET TO BE FURNISHED BY THE CORPS OF ENGINEERS

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation: however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external con-

ditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

SUGAR HILL DAM

PHASE I INSPECTION REPORT

TABLE OF CONTENTS

i. Spillway

1) Type - Broad-crested free overflow with reinforced concrete weir control section.

2) Length of Weir - 150 feet, consisting of one 50-foot weir and one 100-foot weir divided by a stone masonry training wall.

3) Crest Elevation - w/o flashboards 1768 ±
- with flashboards N/A

4) Gates - None.

5) Upstream Channel - Partially riprapped approach which slopes gently upward toward spillway weir.

6) Downstream Channel - Spillway discharges into brush-covered and partially riprapped earth channel excavated in right abutment. Channel contains rock and gabion check dams.

7) General - No comment.

j. Regulating Outlets

1) Outlet Conduit

a) Invert - Intake EL 1717, Discharge EL 1711.

b) Size - 4-foot square.

c) Description - Reinforced concrete conduit with 12-inch thick walls about 300 feet long through the dam with trash rack at the upstream end and a timber bulkhead in an outlet structure at the downstream toe.

d) Control Mechanism
Upstream - An inclined emergency reinforced concrete slide gate 4-feet wide by 5-feet high at an angle of 60 degrees with the horizontal, controlled by a chain winch at the top of the dam.

Downstream - An 8-inch timber bulkhead in the outlet structure at the lower end of the conduit with

2) Flood Control Pool N/A

3) Spillway Crest Pool 1591

4) Top of Dam 1861

5) Test Flood Pool 1912

f. Reservoir Surface (acres)

1) Normal Pool 53

2) Flood Control Pool N/A

3) Spillway Crest Pool 74

4) Top of Dam 80

5) Test Flood Pool 81

g. Dam

1) Type - Earth.

2) Length - 855 feet including spillway.

3) Height
Hydraulic Height - 61 feet.
Structural Height - 63 feet.

4) Top Width - About 16 feet.

5) Side Slopes
Upstream - About 2H:1V.
Downstream - About 2H:1V.
Approximate Volume of Dam - 100,000 cubic yards.

6) Zoning - Central and upstream zones composed of finer borrow material. Downstream zone and upstream slope composed of coarser borrow.

7) Impervious Core - None, see "6) Zoning"

8) Cutoff - 5-foot deep by 15-foot wide central cutoff trench filled with fine-grained borrow.

9) Grout Curtain - None known.

10) Other - No comment.

h. Diversion and Regulating Tunnel - N/A

c. Elevation (feet - NGVD)

Reports and plans submitted by the Central Vermont Public Service Corporation to the Vermont Public Service Commission in the 1930's (see Appendix B3) utilize an arbitrary elevation datum of 100 at the base of the dam. With this elevation datum the spillway crest is at EL 155 and the design top of dam is at EL 160. A recent plan, provided by the Owner for our review during the field inspection, indicates that the spillway crest is at about EL 1768 MSL. Therefore, all elevations used in this report are 1,613 feet greater than those on original reports and plans found in Appendix B3 and are in approximate feet above mean sea level NGVD (National Geodetic Vertical Datum of 1929).

1)	Natural Stream Bed At Toe of Dam - Downstream	1711 ±
	- Upstream	1717 ±
2)	Bottom of Cutoff	1709 ±
	Lowest Foundation Surface (bottom of cutoff)	1709 ±
	Core Wall	None
3)	Maximum Tailwater	Unknown
4)	Normal Pool (site inspection 11/7/79)	1756 ±
5)	Full Flood Control Pool	N/A
6)	Spillway Crest (ungated)	1768 ±
7)	Design Surcharge	Unknown
8)	Top of Dam - low point	1771.5
	- high point	1776.8
	- design	1773
9)	Test Flood Surcharge	1772.1

d. Reservoir (length in feet)

1)	Normal Pool	Est. 2000
2)	Flood Control Pool	N/A
3)	Spillway Crest Pool	2400
4)	Top of Dam	Est. 2500
5)	Test Flood Pool	Est. 2600

e. Storage (acre-feet)

1)	Normal Pool	828

Refer to Section 4 of this report for a complete discussion of operation and maintenance procedures.

1.3 Pertinent Data

a. Drainage Area

1) Location - Central Vermont in northwestern foothills of Green Mountain National Forest.

2) River Basin - Sucker Brook to Lake Dunmore, then to Leicester River, to Otter Creek, to Lake Champlain, to Richelieu River.

3) Shape - Roughly rectangular, about 6000 feet by 13,000 feet.

4) Area - 2.97 square miles, or 1901 acres.

5) Topography - Fairly steep wooded slopes averaging 10% to 20% slope. Elevations vary from EL 1768 to EL 3230.

b. Discharge at Dam Site (cfs)

1) Outlet Works
Outlet conduit, 4-foot square, intake invert EL 1717, discharge capacity 40 cfs at top of dam @ EL 1771.5.

2) Maximum known flood - unknown.

3) Ungated spillway capacity at top of dam, 3030 cfs @ EL 1771.5

4) Ungated spillway capacity @ test flood pool, 3900 cfs @ EL 1772.1.

5) Gated spillway capacity at normal pool - N/A.

6) Gated spillway capacity at test flood pool - N/A.

7) Total spillway capacity at test flood pool, 3900 cfs @ EL 1772.1.

8) Total project discharge at top of dam, 3030 cfs @ EL 1771.5.

9) Total project discharge at test flood pool, 4200 cfs @ EL 1772.1.

Central Vermont Public Service Corporation (CVPS)
77 Grove Street
Rutland, Vermont 05701

Attention: Donald L. Rushford, Esq.
Vice President and General Counsel
(802) 773-2711

f. Operator

Day-to-day operation of the dam is the responsibility of:

J. Douglas Graham, Manager of Hydraulic Generation, CVPS
Edward Lurvey, General Hydraulic Foreman, CVPS

Both can be contacted at:

(802) 773-2711
(Same address as Owner)

g. Purpose of Dam

The dam provides water storage for hydroelectric power generation as a part of the Silver Lake Hydroelectric Development.

h. Design and Construction History

The dam was planned and partially built in 1922 and 1923 by the Hortonia Power Company. The original designer was Vaughan Engineers, of Boston, Massachusetts. It is believed that the firm is no longer in business and the location of its files is unknown. The construction contractor for this original work is not known.

The partially-built dam was purchased by the Central Vermont Public Service Corporation, and plans to raise the dam to its original design height were developed in 1931. It is believed that plans for the raised dam were prepared by the New England Public Service Corporation (NEPSCO). Construction of the raised dam (i.e., the present dam) was completed in 1932 by the Sanders Engineering Company under the direction of F. W. Harris, NEPSCO Civil Engineer.

No other construction, modification, or major repair is known to have occurred. Refer to Section 2 of this report for a complete discussion of the design, construction, and operation history.

i. Normal Operation Procedures

There are no written operation and maintenance procedures for the dam. Maintenance personnel reportedly visit the dam weekly. Also, the Owner indicates that the dam is inspected and reported on annually by a private consultant. The water level in the reservoir presently is maintained below the spillway crest, and outflow from the dam occurs only through the outlet conduit.

The dam also contains a 4-foot by 4-foot reinforced concrete outlet conduit with 12-inch thick walls. The conduit is approximately 300 feet long with a trash rack at the upstream end and a timber bulkhead at the downstream end. An inclined emergency reinforced concrete gate 4-feet by 5-feet high can be used to control flow into the upstream end of the conduit. The gate slides on an angle of 60 degrees with the horizontal and is controlled by a chain winch at the top of the dam. The gate is normally open and can be lowered by its own weight.

At the downstream end of the conduit, there is a reinforced concrete outlet structure. It contains a timber bulkhead across the end of the conduit, as well as two plank baffle weirs in the downstream flume which are used to reduce the velocity of the outflow from the conduit. On the roof of the structure are handwheels for 5 gate valves located in the 8-inch timber bulkhead: two 6-inch valves, two 8-inch valves, and one 10-inch valve. The five valves are used to control outflow from the conduit.

Flow from the outlet structure is discharged into Sucker Brook. Access to the outlet structure is provided by a wooden walkway along the left abutment of the dam.

c. Size Classification

In accordance with recommended guidelines (Reference 1), Sugar Hill Dam is classified as INTERMEDIATE in size because its hydraulic height is 61 feet (within the 40 to 100-foot range) and also because its maximum storage is 1861 acre-feet (within the 1000 to 50,000 acre-foot range).

d. Hazard Classification

In accordance with recommended guidelines (References 1 & 18) involving loss of life and economic loss, Sugar Hill Dam is classified as having a HIGH hazard potential. The dam itself is located in an isolated part of the Green Mountain National Forest and failure of the dam would cause little harm in this area. However, the increase in flow due to a dam failure would damage or destroy Sucker Brook Dam (an earth diversion dam), portions of Branbury State Park, and flood the first floors of about 8 houses along Lake Dunmore to a depth of 4 to 5 feet, with the high flow velocity probably destroying the homes. Damage to a highway bridge on Town Route 53 and to the road on either side of the bridge would be increased by a dam failure. Total economic loss is judged appreciable. Loss of more than a few lives is judged possible. The dam failure analysis is developed in Section 5.5 of this report.

e. Ownership

Since the construction of the dam was completed, the dam has been and is still owned by:

Access to the dam is from State Route 73 to the south via Town roads and trail roads inside the Green Mountain National Forest (see Drainage Area Map, Appendix D-1).

The popular name of the dam is Goshen Dam, and the impoundment is popularly called Sugar Hill Reservoir. The official names are Sugar Hill Dam and Sugar Hill Reservoir. The reservoir is aligned along a northwest - southeast axis with the dam located at the northwesterly end.

The dam is built across Sucker Brook, which is tributary to Lake Dunmore. The nearest downstream community is named Lake Dunmore, population estimated at 50, located about 5 river miles downstream from the dam on the western side of Lake Dunmore, roughly opposite the mouth of Sucker Brook. The community of Lake Dunmore is not an incorporated village, but is simply a group of houses and other structures located in the Town of Salisbury.

b. <u>Description of Dam and Appurtenances</u>

Sugar Hill Dam is a compacted earth embankment of selected fine-grained borrow materials with a single spillway of the free overflow type. The grass and weed-covered embankment is bent slightly upstream where it crosses the natural stream channel. The dam is about 855 feet long (including the spillway) by 61 feet high. The dam crest is about 16 feet wide and both the upstream and downstream slopes are 2H:1V or flatter. The upstream slope and downstream toe are covered with stone riprap.

The dam contains a central cutoff trench 5-feet deep by 15-feet wide filled with the finer borrow material. No impervious core or grout curtain are known to exist. The foundation conditions under the embankment are not known.

The free overflow spillway is an ungated open channel in the right abutment of the dam. The reinforced concrete weir control section is about 3.5 feet below the top of the dam. The weir consists of a 50-foot and a 100-foot section divided by a stone masonry training wall, for a total weir length of 150 feet. Metal flashboard supports exist on the weir crests, but flashboards are presently not used.

The spillway approach channel slopes gently up toward the spillway weir and is riprapped for about 30 feet in front of the spillway weir. The spillway discharge channel is also riprapped for about 30 feet downstream of the weir. The discharge channel is a combination man-made and natural section and is covered with brush. Several rock gabion check dams exist in the channel to slow down discharges and to control erosion. The spillway channel discharges into Sucker Brook several hundred feet downstream from the toe of the dam.

NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

NAME OF DAM: SUGAR HILL DAM, ID NO. VT 176

SECTION 1

PROJECT INFORMATION

1.1 General

a. Authority

The National Dam Inspection Act, Public Law 92-367, August 8, 1972, authorized the Secretary of the Army through the Corps of Engineers to initiate a national program of dam inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Gordon E. Ainsworth and Associates, Inc., has been retained by the New England Division to inspect and report on selected dams in the State of Vermont. Authorization and notice to proceed was issued to Gordon E. Ainsworth and Associates, Inc., under a letter from William E. Hodgson, Jr., Colonel, Corps of Engineers. Contract No. DACW33-80-C-0012 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection

1) Perform technical inspection and evaluation of non-Federal dams to identify conditions which threaten the public, and thus permit correction in a timely manner by non-Federal interests.

2) Encourage and assist the States to initiate quickly effective dam safety programs for non-Federal dams.

3) To update, verify, and complete the National Inventory of Dams.

1.2 Description of Project

a. Location

Referring to the Location and Vicinity Maps at the beginning of this report, Sugar Hill Dam is located in central Vermont in the Town of Goshen, Addison County, about 3 miles South of the community of Bread Loaf. The dam at its maximum section is at Latitude 43 degrees - 54.9 minutes North, Longitude 73 degrees - 0.3 minutes West.

APPROX. SCALE IN MILES

0 2 4 8 12

DATUM - N.G.V.D. 1929, 100' CONTOUR INTERVAL

BASE MAP - 1: 250,000 U.S.G.S. TOPO MAP

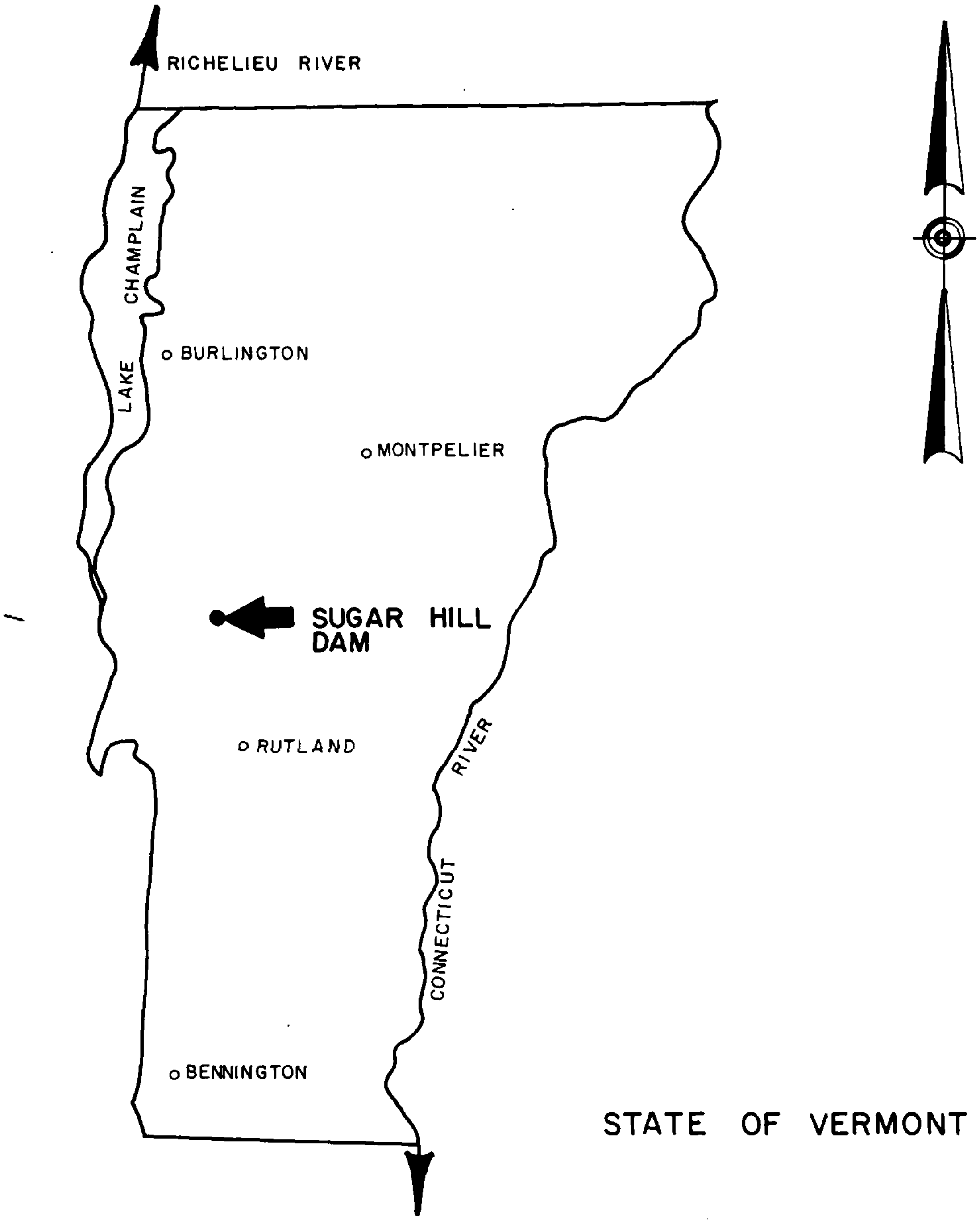
GLENS FALLS, N.Y., VT., N.H., 1956, LIMITED REVISION 1967

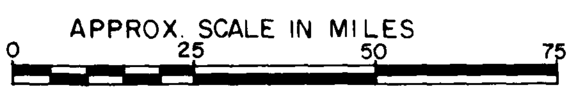
SUGAR HILL DAM VICINITY MAP

GORDON E. AINSWORTH & ASSOCIATES INC.

Engineers, Surveyors and Planners

20 SUGARLOAF ST. SOUTH DEERFIELD MASS. 01373



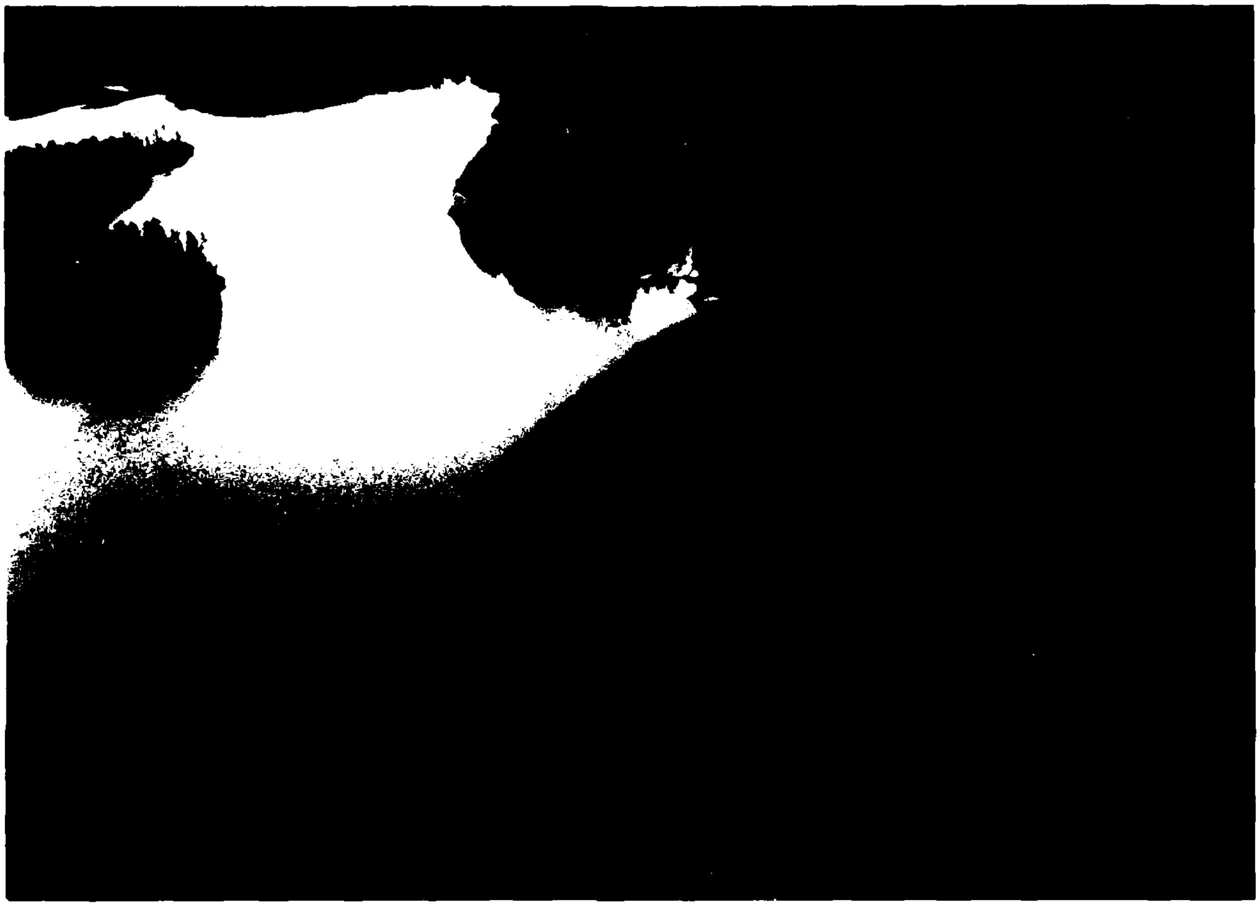


SUGAR HILL DAM LOCATION MAP

GORDON E. AINSWORTH & ASSOCIATES INC

Engineers, Surveyors and Planners

20 SUGARLOAF ST SOUTH DEERFIELD MASS 01373



Overview Photo - Sugar Hill Dam - 11/30/79

APPENDICES

TABLES

five gate valves: two 6-inch, two 8-inch, and one 10-inch. The valves are controlled by handwheels on top of the outlet structure.

e) Other - Outlet conduit can also function as low level drain.

SECTION 2

ENGINEERING DATA

2.1 Design Data

The original dam was designed prior to 1922 by Vaughan Engineers, of Boston, Massachusetts. It is believed that this firm is no longer in business, and the location of its files is unknown.

The original dam was designed for the Hortonia Power Company. Construction of the dam began in 1922, but work was halted in 1923. The partially-built dam was subsequently purchased by Central Vermont Public Service Commission (CVPS, the present Owner). Plans to raise and complete the dam to its original design height appear to have been developed in 1931 by the New England Public Service Corporation (NEPSCO). NEPSCO was thought to be the present New England Power Service Corporation, located at 25 Research Drive, Westborough, Massachusetts 01581, telephone (617) 366-9011. They were contacted, but they indicated that they could find no data on the dam. Subsequently, it was learned that they are not the successors to NEPSCO. The present business status and location of NEPSCO is unknown.

The only available data covering the design and construction of the dam is included in Appendix B3. It consists of a report on construction of the dam (starting on Appendix B3-1), including one record drawing of construction (see Appendix B3-11). This report was prepared by H.K. Barrows, Consulting Engineer of Boston, in 1932 just after construction had been completed, and was requested by the Vermont Public Service Commission (PSC). The PSC order approving the construction contains some additional data and is included starting on Appendix B3-13.

No other design data or drawings were available. The construction specifications were not available.

2.2 Construction Data

a. Initial Construction

Construction of the original dam started in 1922 for the Hortonia Power Company but was halted in 1923. The construction contractor for this original work is not known. After the present Owner purchased the partially-built dam, construction began again in 1931 and was completed in 1932. This construction to raise and complete the dam was done by Sanders Engineering Company under the direction of F.W. Harris, NEPSCO Civil Engineer. The present business status and location of Sanders Engineering is unknown.

Background data on the original construction and the later raising is contained in the report and PSC order discussed previously in Section 2.1 and included starting on Appendix

B3-1. The original construction included the embankment to about 38 feet high, a stone rubble spillway about 50 feet long, a temporary timber spillway at a lower elevation, and the entire length of the outlet conduit. Later, the embankment was raised to its present height of about 60 feet, a 100-foot concrete weir spillway was added, and repairs and improvements to the outlet structure were made.

No other records on the construction of the dam are known.

b. Modifications

No records of any modifications to the dam are known.

c. Repairs and Maintenance

No records of any repairs to the dam are known.

d. Pending Remedial Work

The Owner has no plans for any pending remedial work.

2.3 Operation Data

a. Inspections

Three inspection reports were available and all are included in Appendix B3. The first report (starting on Appendix B3-16) is suspected to have been prepared by Stephen H. Haybrook, on behalf of the State of Vermont, sometime prior to 1953. It contains a brief history and description, but the report is incomplete. Seepage through or under the concrete spillway weir was noted.

The second inspection report starts on Appendix B3-12, and appears to be a subsequent report by Haybrook in 1953. A boil was observed below the downstream toe at maximum section.

The third, and last documented inspection available, is included as Appendix B3-20. It was performed on June 10, 1954, by Byron O. McCoy, of Charles T. Main, Boston, accompanied by two representatives of the Owner. A little seepage at the downstream toe was noted, but there was no sign of the boil that had been reported a year earlier.

The Owner indicates that the dam is inspected annually by the firm of Kleinschmidt and Dutting, Engineering Consultants, 70 Main street, Pittsfield, Maine 04967, telephone (207) 487-3328. However, the Owner did not make the results of those inspections available for review.

b. Performance Observations

There is no instrumentation in the dam. Other than observations made in the inspection reports previously discussed in Section 2.3.a, there are no other known records of performance observations.

c. Water Levels and Discharges

There are no known records of routine water levels and discharges from the dam.

d. Past Floods

There are no known records of past floods at the dam.

e. Previous Failures

There are no known previous failures of the dam.

2.4 Evaluation

a. Availability

As listed on Appendix B1, various engineering data and records are available in the files of the Dam Safety Engineer of the Vermont Department of Water Resources, of the Vermont Public Service Board, and of Vermont Public Records. This data was reviewed, and copies of the records significant to the dam are included in chronological order in Appendix B3. Discussion of the data starts at the beginning of this section of the report. The Owner was unwilling to make their annual inspection reports or other data on file available. The Owner did make one drawing available for review during the field inspection, but the Owner would not allow it to be photographed and would not release it for subsequent review.

b. Adequacy

Available data consisted of a report on construction of the dam just after its completion, including a poor copy of one record drawing of construction, together with copies of three later inspection reports. Such data as the design calculations, construction specifications, data on the foundation and embankment soils, and detailed operation and performance data were not available. The lack of such in-depth engineering data does not permit a comprehensive review. Therefore, the adequacy of this dam could not be assessed with respect to reviewing design, construction, and operation data.

c. Validity

Based on field observation and checking, the limited data available generally appears valid. Some exceptions noted are:

1) Barrow's report after construction in 1932 (see Appendix B3-6) indicates that the regulating valves in the timber bulkhead in the outlet structure consist of "two 6-inch, two 10-inch and three smaller" valves, for a total of 7 valves. The record drawing of construction (see Appendix B3-11) indicates only five valves, consisting of two 6-inch, two 8-inch and one 10-inch. Five stems and handwheels now exist (visible in Photo C-6B). Therefore, this five-valve configuration was assumed correct for this inspection.

2) Original data in Appendix B3 indicate that a level top of dam was intended about 5 feet higher than the spillway crest. Field measurements (see Appendix B2) show a very non-level top of dam with the lowest point only 3.8 feet above the spillway crest.

3) Original data references a spillway crest consisting of a 50-foot long section and another 100-foot long section. Field measurements (see Appendix B2) indicate lengths of 50.6 and 102.0 feet, respectively.

4) The drawing provided by the Owner for our review at the site during our field inspection indicates elevations on the outlet structure that appear to be about 10 feet too low when compared with the elevations (adjusted to the same base) in Barrow's Report (see Appendix B3-6) and on the record drawing (see Appendix B3-11). The cause of this discrepancy is not known. The higher elevations are assumed correct in this report.

SECTION 3

VISUAL INSPECTION

3.1 Findings

a. General

Sugar Hill Dam was inspected on November 7, 1979. The inspection party (see Appendix A-1) was accompanied by two representatives of the Owner: Mr. J. Douglas Graham, Manager of Hydraulic Generation, and Mr. Edward Lurvey, General Hydraulic Foreman. Also present was Mr. Peter Barranco, Jr., Dam Safety Engineer of the Vermont Department of Water Resources. The weather was overcast, and the temperature was about 46° F. The water surface was at about EL 1756, about 12 feet below the spillway crest. The Visual Inspection Checklist is included as Appendix A, while selected photos taken during the inspection are included in Appendix C. Appendix C-1 is a photo index map. The Overview Photo at the beginning of the report as well as several of the photos in Appendix C are aerial photos taken from a helicopter on November 30, 1979.

b. Dam

Photo C-2A is a view along the crest of Sugar Hill Dam from the left abutment, and Photo C-2B is a view from the left training wall of the spillway. The spillway is at the right end of the dam. The upstream slope, showing the boulder riprap, is shown in Photo C-3A. The downstream slope from the left abutment is seen in Photo C-2A. A more detailed view of the downstream slope, taken from a point downstream from the dam, is given in Photo C-3B.

On the downstream side of the dam there is a zone of ponded water, as shown in Photo C-4A. The source of this water is not known. No flow was evident on the day of inspection. Previous inspection reports (see report for inspection of June 10, 1954 signed by the Executive Vice President of the Owner, included as Appendix B3-20) indicate that a boil was observed on the downstream side in the spring of 1953 during high water levels. At some time during the intervening years, it was determined by the Owner that the water level behind this dam would be kept relatively low. As shown on Photo C-3A, the water level on the day of inspection was about 16 feet below the top of dam, or about 12 feet below the spillway crest.

There are several features that were observed on the day of inspection in the vicinity of the outlet conduit that passes through the dam. First, a very thin, barely-visible, crack-like line was noted on the crest near the anchor block for the chain winch mechanism, which is seen in Photo C-5B. This line extends

in a transverse direction across the downstream crestline and then disappears a few feet below crest level. The location of this line is approximately 20 feet to the right of the point at which the conduit passes under the crest of the dam.

Second, the downstream slope of the dam, particularly in the central portion, is "hummocky." Many irregularities that could be due to erosion or frost action were noted.

Third, the downstream slope around and above the outlet structure is eroded. It appears that the soil may be eroding from below the dumped rock (stone loading). Since the outlet structure is cracked, it is also possible that piping may be occurring down into cracks in the conduit in the vicinity of its connection with the outlet structure.

The downstream slope is covered generally with small trees and brush up to 7 feet tall, as may be seen in Photos C-2A and C-3B. Stumps up to 9 inches in diameter are decaying in the downstream slope. Several bare areas, due to surface erosion, are evident. One such area, shown in Photo C-4B, is located near Sta 2+00. Also, some small animal holes were noted in the downstream slope, particularly near the left abutment.

At the upstream left abutment contact line an erosion gully has formed due to surface runoff. This gully appears to be in glacial till.

The crest of the dam is tilted slightly downward on the downstream side, as may be seen in Photo C-5A.

The upstream slope is well covered with a boulder riprap. (See Photo C-3A.) Small brush has been allowed to take root in the riprap in recent years.

c. Appurtenant Structures

1) Intake Structure

The intake structure is a reinforced concrete pipe end section-like structure located about 120 feet upstream of the crest of the dam at about its mid-length. The intake structure was not inspected because it was completely submerged. The intake structure is normally completely underwater at all pool elevations except when the reservoir is almost drained.

There is an emergency reinforced concrete slide gate held open above the intake structure by a chain winch and anchor block located on the dam crest above the intake. The concrete anchor block and chain winch are shown at the left in Photo C-5B. The winch was not operated, but appears in good condition. The concrete anchor block is tilted 2 inches downward on its upstream end relative to its downstream end over its 6-foot length.

2) Outlet Transition and Conduit

The outlet transition and conduit consists of a 4-foot square concrete conduit through the dam from the intake structure to the outlet structure at the downstream toe. The outlet transition and conduit could not be inspected because they were completely full of water. A bulkhead across the downstream end of the conduit inside the outlet structure maintains the outlet conduit full of water, and under pressure, as its normal condition.

3) Outlet Structure and Control Tower

The outlet structure and control tower are together in one reinforced concrete structure located at the downstream toe of the dam to the left of its midpoint. (See Overview Photo and Photos C-3B, C-6A, and C-6B.) The upper level of the structure can be considered a control tower, since it contains the timber bulkhead with gate valves across the end of outlet conduit. The inspection checklist for this portion of the structure is on Appendix A-5. The remaining portion of the outlet structure is on the lower level and is covered by the inspection checklist on Appendix A-7. Only the outside of the outlet structure was inspected. The inside was not accessible due to the flow of water.

The outside condition of the outlet structure is fair. The vertical concrete surfaces, mainly the downstream face, on the upper level (control tower) are in poorer condition with honeycombing visible. Hairline cracks are visible in various places with a major crack at the left front corner of the upper section (see Photo C-7A). Similiar cracking is also evident at the left front corner of the lower section. Part of this cracking is visible at the top center of Photo C-7B. Photo C-7B also shows the badly eroded concrete on the left training wall of the outlet channel under the outlet structure. Similiar damage is also evident on the right training wall.

The five handwheels that control the gate valves in the bulkhead are visible above the upper section of the structure in Photo C-6B. The handwheels were secured with a locked chain and were not operated. Some or all of the valves were open, and/or the bulkhead was leaking, judging from the flow of water from the outlet structure.

4) Service Walkway

As seen in Photo C-6A, the service walkway consists of a stepped, wooden walkway down the slope to the outlet structure. The inspection checklist is on Appendix A-9.

The walkway deck appears structurally sound, but the slightly inclined wooden surface was extremely slippery when only slightly moist. The wooden railing was wobbly and not secure.

5) Spillway and Discharge Channel

The spillway is in the right abutment of the dam. (See Overview Photo and Photo C-11B.) The spillway consists of a very short approach channel, a concrete overflow weir, and an excavated earth discharge channel. The inspection checklist is on Appendix A-8.

The approach channel is essentially the reservoir bottom sloping up to the spillway weir. As seen in Photos C-9A and C-9B, the sand floor of the channel is littered with stone and boulders and with brush and trees up to about 8 feet high. Some logs are also evident.

The spillway weir is pictured in Photos C-8A and C-9A. It consists of 50-foot long crest (foreground of Photo C-9A) and a newer 100-foot crest separated by a concrete and stone masonry training wall. The stone masonry portion of the left training wall is in poor condition. The mortar is deteriorating and individual stones are becoming loosened. (See Photo C-8B.) The central training wall between the spillway weirs is similiarly in poor condition. The concrete weirs themselves appear in good condition. The 100-foot weir has only some hairline cracking. The 50-foot weir does have 3 larger cracks all the way through (the one near the center is visible in the bottom center of Photo C-8A at the flashboard support), but no significant differential movement was observed. The flashboard supports are rusted but, according to the Owner, they are not used and are scheduled to be removed in the near future. The supports in the 50-foot weir will have to be cut off, since they are steel channels embedded in the concrete (see foreground of Photo C-9A), whereas the steel channel supports in the 100-foot weir simply unbolt.

The spillway discharge channel is covered with brush and small trees as seen in Photo C-10A. There are several small rock check dams across the channel that are obscured by vegetation, and one large rock gabion structure a little further downstream where the channel narrows. (See Photo C-10B.) The gabion structure appears to be in only fair condition, with some rocks displaced, the wire very rusted, and some small trees growing out of it. Just downstream of the gabion structure, the discharge channel is fairly heavily forested on its way down to Sucker Brook (see Photo C-11A).

d. Reservoir Area

There does not appear to be excessive reservoir sedimentation. No potential landslide areas were noted around the reservoir. Also, there does not appear to be any potential hazard due to backwater flooding of the reservoir. No features were observed that might cause excessive alteration of the drainage area or increased inflow.

e. Downstream Channel

The downstream channel is the continuation of Sucker Brook starting from the outlet structure. (See Overview Photo.) From the outlet structure to Lake Dunmore, a distance of about 4.7 stream miles, Sucker Brook is generally a rocky, sometimes steep channel that is heavily wooded on both sides. For a map of the downstream channel, refer to the Drainage Area Map, Appendix D-1, which also indexes photos that cover the downstream area. Photo C-11B is an aerial overview of the reservoir and dam looking downstream.

About 2.7 miles downstream from the dam (just below Sta 140+00), Dutton Brook joins Sucker Brook behind the Sucker Brook Diversion Dam, which is also part of the Silver Lake Hydroelectric Development of the Owner. Photo C-12A is an aerial overview of Sucker Brook Diversion Dam looking upstream, showing Sucker Brook entering from the left and Dutton Brook from the right. The spillway is at the extreme left of the photo. Sucker Brook Dam has very little storage capacity and is normally very low or empty since the 4-foot diameter diversion pipe appears large enough to divert all the normal flow from Sucker Brook and Dutton Brook to Silver Lake located to the West. (See separate Phase I Inspection Report on Sucker Brook Dam, VT 00212.)

About 3.3 miles downstream (just below Sta 170+00), Voters Brook joins Sucker Brook. About 3.8 miles downstream (Sta 200+00), an unnamed tributary joins Sucker Brook from the north. Also, approximately at this point any flow from Silver Lake would join Sucker Brook from the south. (See separate Phase I Inspection Report on Silver Lake Dam, VT 00196.)

About 4.3 miles downstream (Sta 229+00), Sucker Brook runs under a bridge on Town Route No. 53 (formerly a State highway, See Photo C-12B). Before reaching the bridge, Sucker Brook drops down from the mountains over so-called Lana Falls. Photo C-13A is an aerial overview looking upstream which shows the mountains in the background and the low-lying area on the shore of Lake Dunmore in the foreground.

Photo C-13B is a closer aerial view of the mouth of Sucker Brook where it flows into Lake Dunmore, and the adjacent low-lying houses and hazard area.

3.2 Evaluation

Based on the geotechnical aspects of the visual inspection, this dam appears to be in fair condition. The erosion that appears to be occurring in the vicinity of the outlet structure may cause serious consequences if the water level in the dam rises. In addition, the thin, crack-like line that was observed crossing the downstream portion of the crest transversely could become a path of seepage if the reservoir level rises. The deterioration

of the left training wall of the spillway could lead to that wall being eroded away during periods when water is flowing over the spillway. The significance of the water ponded downstream is not known.

The remaining features observed, namely, the brush and decaying stumps, the erosion at the upstream left abutment contact line, the erosion and animal holes on the downstream face, all indicate the need for maintenance.

The intake structure and emergency gate were not inspected because they were submerged. They should be inspected. The anchor block for the emergency gate chain winch should be monitored for possible movement.

The outlet conduit was not inspected because it was full of water. It should be dewatered and thoroughly inspected. An excavation should be made down to the conduit just above the outlet structure to determine whether piping into or out of the conduit may be occurring.

The inside of the outlet structure should be similiarly dewatered and inspected. The condition of the 5 regulating gate valves inside should be determined. When dewatered, the major concrete erosion damage to the outlet channel training walls and the apparent cracks in the outlet structure should be better assessed. The best method of repair should be determined in order to upgrade the outlet structure from its present fair condition.

For safety, the wooden service walkway to the outlet structure should be made skid-proof and the railing should be firmly anchored.

The deteriorating stone masonry in the left training wall of the spillway should be repaired. Of lesser importance is the deteriorating stone masonry in the central training wall between the two spillway weirs. Cracking in the older 50-foot long spillway weir should be repaired.

The spillway approach and discharge channels should be cleared of brush, trees, and any logs. The rock check dams and gabion structure in the discharge channel should be maintained. Rock paving just upstream and downstream of the spillway weir should be redistributed, replaced, and maintained.

SECTION 4

OPERATION AND MAINTENANCE PROCEDURES

4.1 Operation Procedures

a. General

Sugar Hill Reservoir has been and still is used as a storage reservoir in the Silver Lake Hydroelectric Power Development. The water level in the reservoir has reportedly been maintained well below the spillway crest in the recent past. At the time of inspection, it was approximately 12 feet below the spillway crest. Several of the valves in the outlet structure are open (unknown as to which ones) and they allow for continuous outflow from the dam into Sucker Brook.

The overflow spillway is ungated and wide open and its flashboards have been removed. The flashboard supports (visible in Photos C-8A and C-9A) are supposed to be removed in the near future according to the Owner. Reportedly the spillway has not operated in the recent past because of the low reservoir level which has been maintained.

There are no written operation procedures for the dam and reservoir.

The Owner indicates that the dam is inspected annually by the firm of Kleinschmidt and Dutting, Engineering Consultants, 70 Main Street, Pittsfield, Maine 04967, telephone (207) 487-3328. However, the Owner did not make the results of those inspections available for review.

b. Emergency Action Plan and Warning System

An emergency action plan with a warning system is in effect for Sugar Hill Dam, according to the Owner. It involves stationing a company employee with a radio at the dam during severe storm events. If an emergency situation develops, he alerts a dispatcher who then informs State Police and local Town officials of the situation.

According to the Owner, the emergency action plan is in writing. However, the Owner would not produce a copy for review or inclusion in this report.

4.2 Maintenance Procedures

a. General

According to the Owner, maintenance crews visit and inspect the dam once a week and perform routine maintenance,

such as brush clearing, annually. There are no written maintenance procedures for the dam and reservoir and their operating facilities.

b. Operating Facilities

(Covered under preceding Section 4.2.a - General.)

4.3 Evaluation

Written operation and maintenance procedures for this dam do not exist. Although routine maintenance of the dam is said to occur annually, our visual inspection suggests that slope maintenance, for instance, has been rather irregular and less often than yearly. Brush growth and tree stumps were evident on the slopes as well as erosion. Effective operation and maintenance procedures need to be developed and implemented by the Owner in order to avoid deterioration of the dam.

As part of the operation procedure, the Owner should formalize the reservoir regulation plan that is now used to maintain normal water level below the spillway crest. This is necessary due to the dam's inadequate spillway capacity when starting with a normal pool at the spillway crest (see Sections 5 and 7), and due to questions about the physical condition of the dam and spillway (see Sections 3, 6 and 7).

SECTION 5

EVALUATION OF HYDRAULICS AND HYDROLOGY

.1 General

Sugar Hill Dam is shown on the Location and Vicinity Maps t the beginning of this report and on the Drainage Area Map, ppendix D-1. The dam and reservoir are located on Sucker Brook n central Vermont. About 14,000 feet downstream of the dam utton Brook enters Sucker Brook behind the Sucker Brook Dam. fter this dam Sucker Brook flows another 11,000 feet before it rains into Lake Dunmore. Lake Dunmore is at the head of the eicester River which runs westward to the Otter Creek. The tter Creek runs northward and flows into Lake Champlain, which n turn is drained to the north by the Richelieu River.

The total drainage area at the dam is about 2.97 square iles, of which about 0.12 square miles (74 acres), or about 4%, s actual reservoir surface at the spillway crest elevation. See Appendices D-1 thru D-3.) Being in the northwestern foothills of the Green Mountain National Forest, the topography is haracterized by fairly steep wooded slopes averaging 10% to 20%. he elevation of the drainage area varies approximately from EL 768 to EL 3230.

.2 Design Data

There are no known records of the hydraulic and hydrologic riteria used in the original design of the dam and reservoir. he engineering data which was available, mainly old design plans, nspection reports and a petition to build the dam, are discussed n Section 2 of this report.

.3 Experience Data

There are no known records of routine water levels and discharges or of past floods at the dam.

.4 Test Flood Analysis

a. Initial Conditions

The U.S. Army Corps of Engineers Hydrologic Engineering enter's Program HEC-1 DB (Reference 3) was used to develop the test lood hydrology and perform the reservoir routing.

The purpose of this analysis was to evaluate the dam and pillway with respect to their surcharge storage and spillway capaity. Accordingly, it was assumed that the water surface was at he spillway crest at the start of the flood routing. Also, it as assumed that the outlet works were closed even though they re normally partially open. It is estimated that the outlet

3 Remedial Measures

a. Operation and Maintenance Procedures

WITHIN ONE YEAR after their receipt of this Phase I spection Report, the Owner should implement the following operion and maintenance procedures:

1) Maintain the pool at its current level or below until such time as the recommendations in Section 7.2 are carried out. Formalize the reservoir regulation plan now used to accomplish this objective.

2) Repair the erosion gully at the upstream left-abutment contact line.

3) Remove all brush and stumps from upstream and downstream slopes, backfill with properly-selected, compacted soils, and cover with erosion protection materials. Remove brush to a distance of about 20 feet downstream from the toeline. Repeat this process annually.

4) Remove brush, trees, and any logs from the spillway approach and discharge channels and repair the channel surfaces. Protect bare spots against erosion. Rock riprap just upstream and downstream of the spillway weir should be redistributed, replaced, and maintained. The rock check dams and gabion structure in the discharge channel should be maintained. Repeat this process annually.

5) Continue with plans to remove the flashboard supports from the spillway weir.

6) Survey the elevation of the anchor block for the emergency gate chain winch annually to determine any long-term trend in suspected movement.

7) Inspect and determine the condition of the 5 regulating gate valves in the timber bulkhead inside the outlet structure. Also, check the bulkhead.

8) Make the wooden service walkway to the outlet structure skid-proof and firmly anchor the railing.

9) Develop and implement effective operation and maintenance procedures to avoid deterioration of the dam.

10) Continue annual technical inspection of the dam with particular attention given to the type of problems and deficiencies noted in this report.

c. Urgency

WITHIN ONE YEAR after their receipt of this Phase I Inspection Report, the Owner should implement the recommendations given in Section 7.2 and the remedial measures given in Section 7.3.

7.2 Recommendations

WITHIN ONE YEAR after their receipt of this Phase I Inspection Report, the Owner should engage a registered engineer qualified in the design of earth dams to do the following work and provide the consequent recommendations. The Owner should implement those recommendations.

a. Inspect the dam again after it has been cleared of brush. Select appropriate backfill for root holes left after removal of roots and stumps (see Section 7.3.a.3).

b. Determine the cause of and/or monitor the water that is ponded on the downstream side of the dam.

c. Investigate the nature and cause of the thin, crack-like line that was noted on the crest of the dam near the anchor block for the gate mechanism.

d. Dewater and inspect the intake structure and emergency gate.

e. Thoroughly inspect the entire outlet conduit and excavate down to the conduit just above the outlet structure to determine whether piping into or out of the conduit may be occurring.

f. Thoroughly inspect the inside and outside of the outlet structure to determine how best to repair the apparent cracking in spots and the major erosion damage to the concrete training walls of the discharge flume.

g. Perform a detailed hydraulic and hydrologic study to better evaluate spillway capacity. Any detailed hydrologic work should take into account upland storage that may exist in the drainage area that would tend to reduce inflow. If necessary, spillway capacity should be increased by new design and construction.

h. Contingent on the results of the detailed hydraulic and hydrologic study, determine what repairs should be made to the left training wall of the spillway, which is deteriorating, and to the older 50-foot long spillway weir, which is cracking.

SECTION 7

ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition

Sugar Hill Dam in FAIR condition. Significant problems include erosion around the outlet structure and at the upstream left abutment contact line, a thin crack-like line transversely crossing the downstream portion of the crest, brush and decaying stumps on the downstream slope, structural deterioration of portions of the outlet structure, and deterioration of the left training wall of the spillway. Also, the spillway approach and discharge channels are becoming overgrown with brush and small trees and are in need of maintenance.

The spillway is INADEQUATE to pass the test flood without overtopping the dam. In accordance with recommended guidelines of the Corps of Engineers, the dam is classified as INTERMEDIATE in size and as having a HIGH hazard potential. Accordingly, a TEST FLOOD equal to FULL PMF (probable maximum flood) is required. The test flood overtops the dam by a maximum of about 0.6 of a foot with duration of overtopping of about 3.7 hours. Peak inflow for the test flood is 4300 cfs. Peak outflow is reduced very little by reservoir routing and is 4200 cfs. Total project discharge capacity at the top of the dam is due only to the spillway (outlet works assumed closed) and is equal to 3030 cfs, or 72% of the test flood peak outflow.

A flood equal to HALF PMF does not overtop the dam, but results in a minimum freeboard of about 0.8 of a foot.

b. Adequacy of Information

This Phase I Inpsection was based primarily on the visual inspection and the hydraulic and hydrologic computations performed, coupled with sound engineering judgement. The visual inspection was done when the pool was quite low, about 16 feet below the top of an approximately 60-foot high dam. Available data consisted of a report on construction of the dam just after its completion, including a poor copy of one record drawing of construction, together with copies of three later inspection reports. Such data as the design calculations, construction specifications, data on the foundation and embankment soils and detailed operation and performance data were not available. The lack of such in-depth engineering data does not permit a comprehensive review. Therefore, the adequacy of this dam could not be assessed with respect to reviewing design, construction, and operation data.

mine whether this ponded water is seepage through the dam. The area should be inspected carefully after the ground is cleared and monitoring points should be installed for this purpose.

The left training wall of the spillway is gradually losing its mortar. Thus, any high flows through the spillway could easily undermine the wall, wash it away, and begin eroding the dam.

6.2 Design and Construction Data

Nothing of significance relating to structural stability was noted in the limited design and construction data available.

6.3 Post-Construction Changes

In the report of an inspection on June 10, 1954 (see Appendix B3-20), reference was made to former inspections, in 1953, when boils were observed. In the June 10, 1954 report it was indicated that the boil did not seem to be related to the reservoir level, in that the boil gradually "dropped off as the groundwater table in the general area."

During subsequent years, the Central Vermont Public Service Corporation has operated the dam with the reservoir at rather low levels, about 16 feet below the top.

Based on the visual observations and on the former existence of boils, it appears prudent to maintain the pool at its present level, or below, until such time that these past experiences and the present observations can be investigated.

6.4 Seismic Stability

This dam is in Seismic Zone 2 and, therefore, according to recommended guidelines (Reference 1), a seismic stability analysis is not warranted.

SECTION 6

EVALUATION OF STRUCTURAL STABILITY

6.1 Visual Observations

The visual observations that indicate concern about the structural stability of this dam are: (a) the presence of a thin, crack-like line that was noted passing transversely across the downstream portion of the crest, (b) erosion that is occurring on the downstream slope beneath the boulder riprap near the outlet structure, (c) the left training wall of the spillway, and (d) the water that is ponded downstream.

The thin, crack-like line was traced over the downstream crestline until it disappeared a few feet below the crest. Although it is a definite feature, it was not clear whether it was a thin path of erosion, or whether erosion due to surface runoff was occurring along a hairline crack. Since this line appears about 20 feet to the right of the location where the conduit passes through the dam, one might suspect that the feature is related to this buried structure. Careful excavation into the crest is required to determine its cause.

The erosion that is observed just upstream from the outlet structure may be caused by surface runoff. The runoff naturally concentrates near the deepest point of the valley. It can be assumed that the boulder riprap was placed on the embankment without a suitable filter. Therefore, surface runoff would lead to undermining of the stones. However, the outlet structure is cracked and deteriorating. Therefore, buried cracks may exist in the upstream portion of the outlet structure, e.g., at the connection with the conduit. In such a case, paths of seepage could be developing within the dam and piping fines into the cracks during periods of high water.

The control of outflow from the dam is on the downstream end, at the outlet structure. Thus, there is a pressure head within the conduit which may exceed the weight of the overlying soil just above the outlet structure. Any cracks in the conduit could be pathways for flow from the conduit into the embankment. Thus, internal erosion could occur.

Based on the above discussion of possible explanations for the observations, it is necessary to investigate whether the outlet structure is intact near its upstream end and to determine whether the obvious erosion will cause difficulty when the pool is at high levels.

Some ponded water was noted on the downstream side of the dam. The significance of this water is not known. Since boils had been observed in the past, it is necessary to deter-

floods an area about 615 feet wide. Flow accelerates about 3.4 times to 31 fps. The highway bridge, which is visible in Photo C-12B, has an estimated capacity (Reference 17) of only 1000 to 1500 cfs with headwater 8 feet deep (i.e., water level with the road), which is less than even the prior flow of 3030 cfs. Therefore, the increase in flow due to the dam failure would only worsen the already out-of-channel and over-the-roadway flow condition that would exist just prior to the failure.

At Sta 237+00 near houses along Lake Dunmore, peak flow increases about 30 times to 92,000 cfs after about 21 minutes. This causes the water to rise from 2.4 to 7.3 feet deep, an increase of 4.9 feet, which floods an area about 910 feet wide. Flow accelerates about 3.2 times to 19 fps. Ground around the houses is estimated at EL 580 with the first floors estimated at EL 581. Prior flow at EL 580.4 appears not to flood the first floors. The 4.9-foot increase due to the dam failure appears to flood the first floors to a depth of 4 to 5 feet. The 19 fps flow would probably destroy the structures. It is estimated that about 8 houses would be involved in this flooding, plus miscellaneous outbuildings. An adjacent State park would also be flooded and damaged.

The flood routing was not carried any further downstream than Sta 237+00 because Sucker Brook drains into Lake Dunmore just after this station. Lake Dunmore has a surcharge storage capacity of over 1035 acre-feet per foot as compared to the total volume of Sugar Hill Reservoir at the top of dam of 1861 acre-feet. Thus, it appears that the failure of Sugar Hill Dam would cause the level of Lake Dunmore to rise, but how much of a rise, considering the outlet capacity of Lake Dunmore, and what significance the rise would have is unknown without further study.

Thus, it appears that the increase in flow due to a failure of the dam would damage or destroy Sucker Brook Dam (an earth diversion dam), portions of Branbury State Park, and flood the first floors of about 8 houses along Lake Dunmore to a depth of 4 to 5 feet, with the high flow velocity probably destroying the homes. Damage to a highway bridge on Town Route 53 and to the road on either side of the bridge would be increased by a dam failure. Total economic loss is judged appreciable. Loss of more than a few lives is judged possible. Therefore, according to recommended guidelines (Reference 1), the dam is classified as having a high hazard potential.

From the computer listing and plot of the breach hydrograph on Appendices D-30 and D-31, note that the standard calculation interval selected (1 minute = 0.017 hours) was short enough to permit the interpolated breach hydrograph at the standard time interval to closely approximate the computed breach hydrograph. Only the interpolated breach hydrograph is routed downstream.

Appendix D-32 is a computer plot of the complete inflow and outflow hydrograph during and after the breach.

c. Hazard Evaluation

For a sudden major dam failure, BREACH AT TOP OF DAM, the computed maximum water surface elevation for each downstream station is tabulated in Table 5.2 and is plotted on each cross section starting on Appendix D-16. The top widths of flow determined from each cross section are tabulated in Table 5.2 and are plotted on Appendix D-1 to define the limit of the hazard area, i.e. the limit of flooding due to the dam failure. Also, the computed water surface is shown on the channel profile, Appendix D-21.

The average velocity of peak flow (flow divided by total flow area) is also listed in Table 5.2 for each downstream station for both flow cases. For the dam breach case, the flow area calculation is shown on each cross section plot starting on Appendix D-16, and consists of storage for the channel reach defined by the cross section divided by reach length. The channel storage was computed by the HEC-1 DB program for both flow cases.

Just prior to the dam breach, outflow from the dam was 3030 cfs and flow at the first section 700 feet downstream was about 5.5 feet deep at about 10 fps. Approximately 14 minutes after the breach starts, peak outflow from the dam increases about 42 times to 127,000 cfs, causing water 700 feet downstream to rise from 5.5 to 20.2 feet deep, an increase of 14.7 feet, which floods an area about 505 feet wide. Flow accelerates about 2.3 times to 23 fps.

At Sta 140+00 at Sucker Brook Dam, peak flow increases about 31 times to 93,000 cfs after about 19 minutes. This causes the water to rise from 3.6 to 13.0 feet deep, an increase of 9.4 feet, which floods an area about 320 feet wide. Flow accelerates about 2.8 times to 31 fps. The increase in flow due to the dam failure would undoubtedly damage or destroy Sucker Brook Dam. Sucker Brook Dam is an earth embankment diversion dam having a maximum storage capacity of about 50 acre-feet and a total discharge capacity of about 4300 cfs (see separate Phase I Inspection Report on Sucker Brook Dam, VT 00212).

At Sta 229+00 at the highway bridge on Town Route 53 (formerly a State highway), peak flow increases about 30 times to 92,000 cfs after about 20 minutes. This causes the water to rise from 2.5 to 7.0 feet deep, an increase of 4.5 feet, which

TABLE 5.2

SUGAR HILL DAM

DAM FAILURE ANALYSIS

CONDITIONS - Top of Dam Elev. 1771.5 (lowest point of non-level top)
Spillway Crest Elev. 1768
Total Project Discharge Capacity at Top of Dam = 3030 cfs ±
due to Spillway only. Outlet Works closed.

	Approx. Peak Flow (cfs)	Time to Peak Flow (hours)	Approx. Max. Water Surface			
			Elev. (feet)	Depth (feet)	Top Width (feet)	Avg. Vel. (fps)
PRIOR FLOW AT TOP OF DAM						
Inflow = Outflow = Total Project Discharge Capacity at Top of Dam						
Start Routing at Top of Dam						
Dam	3030	0.0	1771.5	54.5	--	--
Sta 7+00	3030	0.20	1715.5	5.5	--	10
Sta 140+00 Sucker Brook Dam	3030	2.58	1300.6	3.6	--	11
Sta 224+00	3030	2.67	643.4	3.4	--	56
Sta 229+00 Hwy Bridge	3030	2.68	600.5	2.5	--	9
Sta 237+00 Houses	3030	2.72	580.4	2.4		6
BREACH AT TOP OF DAM						
Inflow = Prior Flow						
Start Routing at Top of Dam						
Start Breach W.S. at Top of Dam						
Time of Failure = 0.00 hour						
Breach Time = 0.23 hour						
Breach Width = 190 feet						
Breach Depth = 54.5 feet						
Trapezoid, 0.5 H : 1V side slopes						
Dam	127,000	0.23	1771.5	54.5	--	--
Sta 7+00	125,000	0.22	1730.2	20.2	505	23
Sta 50+00	110,000	0.25	1547.4	7.4	1730	18
Sta 100+00	98,000	0.28	1446.9	16.9	420	30
Sta 140+00 Sucker Brook Dam	93,000	0.32	1310.0	13.0	320	31
Sta 1[illegible]0+00	93,000	0.32	1141.1	21.1	125	62
Sta 200+00	92,000	0.33	905.6	10.6	235	54
Sta 216+00	92,000	0.33	823.8	23.8	100	68
Sta 224+00	92,000	0.33	655.3	15.3	125	89
Sta 229+00 Hwy Bridge	92,000	0.33	605.0	7.0	615	31
Sta 237+00 Houses	92,000	0.35	585.3	7.3	910	19

To model a sudden major dam failure, maximum breach geometry was selected as follows: constant trapezoidal shape with moderate 0.5H:1V side slopes, breach width across the bottom of the trapezoid equal to about 40% of the dam length at mid height (approximately 190 feet), and a breach depth below the low point of the top of the dam equal to 54.5 feet, which approximates a full depth failure that would completely drain the reservoir. Breach geometry is illustrated on Appendix D-27.

Breach time, or time for the breach width to progress from the top to the bottom of the dam, was selected so that the peak outflow using the HEC-1 DB program would approximate that computed by the Corps of Engineers' "Rule of Thumb" method using the same breach width and depth. The selection of breach time is shown on Appendix D-27. Rule of Thumb peak outflow is 128,500 cfs. A breach time of 0.23 hours, or 13.8 minutes, was selected for the HEC-1 DB program, which results in a peak outflow of about 127,000 cfs.

The inputted cross sections defining average downstream channel reaches were developed from and are located on the USGS map included as Appendix D-1. Hand plottings of the cross sections start on Appendix D-16, while Appendix D-21 is a profile of the downstream channel. Normal depth channel routing was performed by the HEC-1 DB program using the Manning's n values for left overbank, channel, and right overbank, as listed on each cross section plot. The overbank points and the actual channel section in between are only an approximation of the true natural channel. This is because of the constraints of the small scale USGS map that the cross sections were developed from and of the limited 8-point cross section accepted by the program. The third and sixth point on each cross section are defined as the overbank points. Therefore, distinguishing between in-channel and overbank flow cannot be done reliably by simple comparison of the water surface depth with the defined overbank points. It must be done by judging the calculated quantity, depth, width, and velocity of flow against the real channel cross section as it exists.

b. Results of Analysis

The results of the dam failure analysis using the HEC-1 DB program are summarized in Table 5.2. PRIOR FLOW AT TOP OF DAM establishes initial conditions downstream due to steady state total project discharge capacity at the top of dam with no dam breach. The computer input and selected pages of the computer output start on Appendix D-22. In Table 5.2 only the results at the more important stations are summarized.

BREACH AT TOP OF DAM is a major sudden failure of the dam under the conditions previously discussed in Section 5.5.a. Results are summarized in Table 5.2 for all stations, with the computer input and selected pages of the computer output starting on Appendix D-28.

The peak portion of the inflow and outflow hydrograph for the test flood of full PMF is shown by the computer plot on Appendix D-13. Total project discharge capacity at the top of the dam is due only to the spillway (outlet works assumed closed) and is equal to 3030 cfs, or 72% of the test flood peak outflow.

Since the test flood of full PMF overtops the dam and the dam is classified as having a high hazard potential, a flood equal to half PMF was evaluated as required by the Corps. This flood was modeled as half of full PMF total runoff, and appears as the second ratio of the PMF in the overtopping analysis computer input and output starting on Appendix D-9. The results are summarized in Table 5.1. A flood of half PMF does not overtop the dam, but results in a minimum freeboard of about 0.8 of a foot.

As indicated by the footnote (e) on Table 5.1, the possible effect of the outlet works being open was investigated by a second routing of the test flood. The resulting maximum water surface is the same as when the outlet works are assumed open. The computer input and output for this routing are not included in this report. The outlet works were modeled by the HEC-1 DB program as an orifice having an area equal to the combined areas of the 5 regulating gate valves in the outlet structure.

As indicated by footnote (f) on Table 5.1, the possible effect of taking the spillway crest at the reported exact EL 1767.7 rather than the approximate EL 1768 used in the test flood analysis was investigated by a third routing of the test flood. The resulting maximum water surface elevation is 0.2 of a foot lower than with the spillway crest at EL 1768, but the test flood still overtops the dam by 0.4 of a foot. The computer input and output for this routing are not included in this report.

5.5 Dam Failure Analysis

a. Failure Conditions

In order to evaluate the downstream hazard, the flow just prior to and then due to an assumed major failure or breach of the dam was routed downstream using the HEC-1 DB program. Stream conditions just prior to and after the assumed failure were compared. Corps of Engineers' criteria call for breaching the dam with no inflow flood and with the water surface static at the top of the dam, or static at the test flood pool if the test flood does not overtop the dam. Since the overtopping analysis shows that the test flood does overtop the dam, the dam breach was begun at time zero with the water surface at the top of the dam. The contents of the reservoir were routed through the breach as the breach progressed.

TABLE 5.1

SUGAR HILL DAM

OVERTOPPING ANALYSIS

CONDITIONS -

Total Drainage Area = 2.97 Square Miles (f)
Start Routing at Spillway Crest Elev. 1768
Top of Dam Elev. 1771.5 (lowest point of non-level top)
Total Project Discharge Capacity at Top of Dam 3030 cfs ± due to Spillway only. Outlet Works Closed (e)
Some Values Rounded from Computed Results

		TEST FLOOD FULL PMF	HALF PMF (a)
INFLOW			
24-hour Rainfall (inches)		18.5	10.6 (b)
24-hour Rainfall Excess (inches) (c)		15.9	7.9 (c)
Peak Inflow	(cfs)	4300	2150
	(csm)	1448	724
OUTFLOW			
Peak Outflow	(cfs)	4200	2010
	(csm)	1414	677
Time to Peak Outflow (hours)		19.08	19.33
Maximum Storage (acre-feet)		1912	1794
Max. W.S. Elevation (feet-NGVD)		1772.1 (e,f)	1770.7
Minimum Freeboard (feet)		overtopped	0.8
Maximum Depth over Dam (feet)		0.6	not overtopped
Duration of Overtopping (hours)		3.67	n/a

(a) One-half of full PMF total runoff, including base flow. For full PMF base flow = 2 cfs per square mile = 6 cfs ±
(b) Approximation assuming total losses are the same as for the full PMF.
(c) Rainfall Excess = Rainfall for the Reservoir Surface. For the rest of the drainage area, losses are assumed to be 1.0 inch initially and 0.1 inch per hour thereafter.
(d) Equal to one-half of full PMF value.
(e) For outlet works open, total discharge capacity = 3070 cfs ± and maximum W.S. Elevation = 1772.1 for test flood .
(f) For start of routing at spillway crest Elev. 1767.7, total discharge capacity = 3430 cfs ± and maximum W.S. Elevation = 1771.9 for test flood.

e. Development of Test Flood ?

The index PMP (probable maximum precipitation) inputted to the HEC-1 DB program was 17.5 inches for a 24-hour duration, all-season storm over a 200 square mile basin, according to HMR 33 (Reference 4). Maximum 6-hour, 12-hour, and 24-hour precipitation for the actual size of the drainage area (same for 10 square miles or less) were inputted to the program as percentages of the index PMP in accordance with HMR 33. A storm reduction coefficient was then applied internally by the program in order to transpose or center the storm over the actual total drainage area. Thus, the corrected 24-hour PMP for the actual total drainage area became 18.5 inches.

All precipitation was distributed by the program using the Standard Project Storm arrangement of EM 1110-2-1411 (Reference 13), including the percentage distribution for the maximum 6-hour precipitation, and by both the arrangement and percentage distribution from HYDRO-35 (Reference 6) for the maximum 1-hour precipitation.

Appendix D-8 summarizes the subarea, loss rate, and unit hydrograph data inputted to the program. Only two subareas were used. Subarea 1 consists of all the drainage area around the reservoir, and Subarea 2 consists of just the reservoir surface. For the land in Subarea 1, loss rates were assumed to be 1.0 inch initially and a constant 0.1 inch per hour thereafter. Snyder unit hydrograph parameters were assumed for average conditions per Appendix D-8 and inputted to the program. A conservative standard lag time was used. The program uses the inputted Snyder coefficients to solve by iteration for approximate Clark coefficients, which are then used to calculate the runoff hydrograph.

For the reservoir surface making up Subarea 2, loss rates were set to zero so that rainfall would equal rainfall excess, or runoff. Assuming no delay in the rainfall/runoff response, a constant unit hydrograph for a rainfall duration equal to the HEC-1 DB calculation interval was developed per Appendix D-8 and inputted to the program.

f. Overtopping Potential

The results of the overtopping analysis using the HEC-1 DB program are summarized in Table 5.1. The overtopping analysis computer input and complete output for the test flood of full PMF are included starting on Appendix D-9.

As noted from Table 5.1, the test flood of full PMF overtops the dam by a maximum of about 0.6 of a foot with duration of overtopping of about 3.7 hours. Peak infl[illegible] for the test flood is 4300 cfs, or 1448 csm (cfs per square mile). Peak outflow is reduced very little by reservoir routing to 4200 cfs, or 1414 csm, and occurs about 19.0 hours after the start of the storm.

The discharge capacity of the spillway was computed assuming critical flow over a rectangular broad-crested weir. The crest elevation, length, appropriate discharge coefficient, and exponent of head were inputed to the HEC-1 DB program (Reference 3). The formula used for the calculation, as well as the results of hand calculation at selected points, is shown on Appendix D-6. With water 3.5 feet over the spillway (water level at the low point of dam crest), the spillway has a discharge capacity of about 3,030 cfs.

Taking the spillway crest at EL 1768 and the dam crest at EL 1771.5, total discharge computations are summarized on Appendix D-6 and graphed on Appendix D-7. Total discharge from the dam is the sum of the discharges from the spillway plus flow over the dam for the overtopping condition. As discussed previously in Section 5.4.a, the outlet works were assumed closed for this analysis and not contributing to the total discharge capacity. Flow over the dam was computed by the HEC-1 DB program, assuming critical flow over a non-level dam crest, using inputted crest length and elevation data (see Appendix B2). The computed results are hand tabulated on Appendix D-6.

With the reservoir at the low point of dam crest, EL 1771.5, 3.5 feet over the spillway crest, the total discharge from the dam is about 3030 cfs. This is due solely to the spillway. Also, with an average discharge of about 1,515 cfs over the 3.5-foot depth from the top of the dam down to the spillway crest, it would take about 2.2 hours for the spillway to drain the 270 acre-feet of storage between the top of the dam and the spillway crest, or about 37 minutes per foot, all assuming no inflow.

d. Selection of Test Flood

Based on the dam failure analysis presented later in Section 5.5, Sugar Hill Dam is classified as having a high hazard potential (increase in flow due to a dam failure would result in appreciable economic loss and possible loss of more than a few lives caused by damage or destruction of an earth diversion dam and portions of Branbury State Park, an increase in damage to a highway bridge on Town Route 53 and the road on either side of the bridge, and flooding of the first floors of about 8 houses along Lake Dunmore to a depth of 4 to 5 feet, with the high flow velocity probably destroying the homes). Since the dam is also classified as intermediate in size (see Section 1.2.c), recommended guidelines of the Corps of Engineers (Reference 1) indicate a test flood equal to the full PMF. Therefore, the test flood selected for this evaluation was the full PMF (per Table 5.1, peak inflow = 4300 cfs, peak outflow = 4200 cfs).

The PMF event is that hypothetical flood flow produced by the most critical combination of precipitation, minimum infiltration loss, and concentration of runoff that is considered reasonably possible for a particular drainage area.

works have a discharge capacity of about 40 cfs when they are fully open and the water surface is at the top of the dam. This is so small compared to the discharge capacity of the spillway at the top of the dam of about 3030 cfs (see Section 5.4.c), that assuming the outlet works are either fully closed or open makes no significant difference in the maximum water surface elevation caused by the test flood (see Section 5.4.f).

A constant base flow of 2 cfs per square mile was chosen to represent average conditions in the drainage area and was inputted into the program for all subareas.

b. Storage Capacity

Stage versus area information for the reservoir was found in the files of the Vermont Dam Safety Engineer (see Appendix B3-21). Using these areas, the capacity of the reservoir was computed by the method of conic sections. The computations were done by the HEC-1 DB program with the results on Appendices D-12 and D-15. A hand tabulation of the elevation - area input and the computed capacity is on Appendix D-2.

Using the computed values, stage-area and stage-storage curves are presented on Appendices D-4 and D-5, respectively. At the spillway crest, EL 1768, the reservoir has a surface area of 74 acres and a total capacity of 1591 acre-feet, or about 518.5 million gallons. At the dam crest, EL 1771.5, the surface area increases to about 80 acres and the capacity to 1861 acre-feet, or about 606.4 million gallons. Surcharge storage between the spillway crest and the dam crest amounts to 270 acre-feet, or about 1.70 inches of runoff from the 2.97 square-mile drainage area. Therefore, the reservoir has some capacity to attenuate peak inflow.

c. Discharge Capacity

The only spillway for the dam is a broad-crested free overflow spillway in the right abutment. Referring to the engineering data in Appendix B and Photos C-8A, C-9A, C-9B, and C-10A, the spillway consists of an approach section, two separate concrete overflow weir control sections, and an excavated earth discharge channel down to the natural stream bed. The overflow weir control sections are 50 and 100 feet wide and consist of concrete weirs with stone masonry training walls. The weir crests are about 3.5 feet lower than the lowest point on the dam crest. The discharge channel narrows after the control weir and becomes fairly steep. It contains several rock and gabion check dams to control discharge velocities and is a grass and brush-covered channel. For this investigation, the effective crest length of the weir control section was approximated as 150 feet, neglecting the stone masonry training wall between the two weirs.

11) Make any improvements necessary in the existing emergency action plan and warning system to ensure proper and timely action during critical periods.

7.4 Alternatives

No practical alternatives exist to the recommendations and remedial measures contained in this report.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECKLIST
DAM INSPECTION

DAM Sugar Hill Dam | DATE November 7, 1979

ID NO. VT 00176 | TIME 0930 - 1245

TOWN Goshen | WEATHER Overcast, 46° F

COUNTY Addison | W.S. ELEV. 1756.1 UPSTREAM

STATE Vermont | 1711 ± DOWNSTREAM

INSPECTION PARTY	RECORDER (X)
1. Thomas Bennedum, Gordon E. Ainsworth & Associates, Inc.	X
2. Edwin Vopelak, Jr., Gordon E. Ainsworth & Associates, Inc.	
3. John Kenworthy, Gordon E. Ainsworth & Associates, Inc.	
4. Steve J. Poulos, Geotechnical Engineers, Inc.	X
5. Peter Barranco, Jr., Vermont Department of Water Resources	
6. J. Douglas Graham, Manager of Hydraulic Generation	
7. Central Vermont Public Service Corporation	
8. Edward Lurvey, General Hydraulic Foreman	
9. Central Vermont Public Service Corporation	
10.	

PROJECT FEATURE/DISCIPLINE	INSPECTOR	REMARKS
1. H & H	T. Bennedum	-
2. Geotechnical	S. Poulos	-
3. Structural	T. Bennedum	-
4. Mechanical	T. Bennedum	-
5. Electrical	None	N/A
6.		

VISUAL INSPECTION CHECKLIST

2 PROJECT Sugar Hill Dam — DATE Nov. 7, 1979

PROJECT FEATURE - — NAME -

DISCIPLINE Geotechnical — NAME S. J. Poulos

	AREA EVALUATED	CONDITION
	DAM EMBANKMENT	
	1 Crest Elevation	EL 1771.5
	2 Current Pool Elevation	EL 1756.1
	3 Maximum Impoundment to Date	Unknown
GEI	4 Surface Cracks	One crack or erosion feature opposite intake structure for conduit.
GEI	5 Pavement Condition	No pavement. Bare dirt road.
GEI	6 Movement or Settlement of Crest	Upstream edge of crest, in car track, is slightly higher than downstream edge. Crest is low from about Sta 5+70 to 7+00.
GEI	7 Lateral Movement	None observed.
GEI	8 Vertical Alignment	See item 6.
GEI	Horizontal Alignment	Arched upstream from left abutment to 4+00.
GEI	Condition at Abutment and at Concrete Structures	Right abutment (LEFT SPILLWAY WALL): OK ANCHOR BLOCK FOR LOW LEVEL INTAKE GATE: Tilted down 2" on upstream end over 6' length. Otherwise OK. LEFT ABUTMENT UPSTREAM: Erosion gully in till at contact. 3-in.-dia. animal hole to right of gully 7' down from crest. Hole about 8" deep. LEFT ABUTMENT DOWNSTREAM: OK. OUTLET STRUCTURE: Erosion upstream of structure, around it, and down into outlet channel. May also be penetrating holes in spalled concrete at downstream end of outlet structure. Unfiltered boulder riprap is shielding view of erosion.
GEI	Indications of Movement of Structural Items on Slopes	None observed.
GEI	Trespassing on Slopes	Free access. Deer hunters.
GEI	Sloughing or Erosion of Slopes or Abutments	DOWNSTREAM SLOPE: Very rough on upper part of downstream face. Hummocks varying ± 1 ft in elevation. About 10 erosion channels downslope up to 1' deep, mostly 6" deep. Eroded bare spot at 2+00 from midslope almost down to toe. Sta 3+10: small 3'x3' delta of silt caught in riprap.

VISUAL INSPECTION CHECKLIST

2A PROJECT Sugar Hill Dam DATE Nov. 7, 1979

PROJECT FEATURE - NAME -

DISCIPLINE Geotechnical NAME S. J. Poulos

	AREA EVALUATED	CONDITION
	<u>DAM EMBANKMENT</u> (Continued)	
GEI	Sloughing or Erosion of Slopes or Abutments (Continued)	2+80 - bottom half of riprap: 3'-4' deep erosion gully probably due to loss of fines under riprap and/or piping into conduit which is immediately below.
GEI	Rock Slope Protection-Riprap Failures	Downstream very irregular. Dumped rock looks as though it filled old erosion gullies.
GEI	Unusual Movement or Cracking at or Near Toe	None observed.
GEI	Unusual Embankment or Downstream Seepage	None observed.
GEI	Piping or Boils	None observed.
GEI	Foundation Drainage Features	None.
GEI	Toe Drains	None.
GEI	Instrumentation System	None.
GEI	Vegetation	Downstream - grass and moss under low brush up to 7' high. Rotted stumps to 9-in.-dia.

VISUAL INSPECTION CHECKLIST

DAM Sugar Hill Dam DATE November 7, 1979 4

DISCIPLINE Structural/H & H INSPECTOR T. Bennedum

DISCIPLINE Geotechnical INSPECTOR S. J. Poulos

AREA EVALUATED	CONDITION
OUTLET WORKS - INTAKE CHANNEL AND INTAKE STRUCTURE	
a. Approach Channel	
Slope Conditions	Forested. Beach slopes are till.
Bottom Conditions	Underwater.
Rock Slides or Falls	None.
Log Boom	None.
Debris	None observed.
Condition of Concrete Lining	N/A
Drains or Weep Holes	N/A
b. Intake Structure	Not observable. Intake is totally underwater.
Condition of Concrete	
Stop Logs and Slots	
	Concrete anchor block and chain winch on dam crest to control emergency concrete slide gate over intake. Winch was not operated, but appears in good condition. Anchor block is tilted down 2" on upstream end over its 6' length.

VISUAL INSPECTION CHECKLIST

DAM Sugar Hill Dam DATE November 7, 1979 5

DISCIPLINE Structural/Mechanical INSPECTOR T. Bennedum

DISCIPLINE No Geotechnical Features INSPECTOR -

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER	Considered as upper level of outlet structure at D/S toe. Outside inspected. Inside not accessible.
a. Concrete and Structural	
General Condition	Fair to poor. Vertical faces, mainly front, show honeycombing.
Condition of Joints	No joints observed.
Spalling	Some on top, mainly R rear corner.
Visible Reinforcing	None.
Rusting or Staining of Concrete	Moss and runoff stained. No rust.
Any Seepage or Efflorescence	Minor efflorescence.
Joint Alignment	N/A
Unusual Seepage or Leaks in Gate Chamber	Not accessible for observation.
Cracks	Major vertical cracks in left front corner. Minor hairline cracks various places.
Rusting or Corrosion of Steel	None observed.
b. Mechanical and Electrical	
Air Vents	None.
Float Wells	None.
Crane Hoist	None.
Elevator	None.
Hydraulic System	None.
Service Gates	5 gate valves in timber bulkhead. Handwheels chained together on top, not operated.
Emergency Gates	
Lightning Protection System	None.
Emergency Power System	None.
Wiring and Lighting System	None.

VISUAL INSPECTION CHECKLIST

DAM Sugar Hill Dam DATE November 7, 1979

DISCIPLINE Structural/H & H INSPECTOR T. Bennedum

DISCIPLINE No Geotechnical Features INSPECTOR -

AREA EVALUATED	CONDITION
OUTLET WORKS - TRANSITION AND CONDUIT	Not observable. Consists of 4-foot square concrete outlet conduit through dam from intake to outlet structure.
General Condition of Concrete	
Rust or Staining on Concrete	
Spalling	
Erosion or Cavitation	
Cracking	
Alignment of Monoliths	
Alignment of Joints	
Numbering of Monoliths	

VISUAL INSPECTION CHECKLIST

DAM Sugar Hill Dam DATE November 7, 1979

DISCIPLINE Structural/H & H INSPECTOR T. Bennedum

DISCIPLINE Geotechnical INSPECTOR S. J. Poulos 7

AREA EVALUATED	CONDITION
OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL	Considered as lower level of outlet structure.
General Condition of Concrete	Top fair. Inside of discharge poor.
Rust or Staining	Outside moss and runoff stained and rust stained at outlet.
Spalling	Yes, at damaged training walls and minor on top.
Erosion or Cavitation	Major damage @ bottom of R & L training walls of discharge.
Visible Reinforcing	Yes, at damaged training walls.
Any Seepage or Efflorescence	Minor efflorescence.
Condition at Joints	No joints observed. Crack at left front corner.
Drain holes	None found.
Channel	
Loose Rock or Trees Overhanging Channel	Heavily forested and over-hanging trees both sides. No loose rocks.
Condition of Discharge Channel	Fair. Some debris and wood which may form dam in high water.
	Only outside of structure inspected. Inside not accessible due to flowing water.

VISUAL INSPECTION CHECKLIST

DAM Sugar Hill Dam DATE November 7, 1979 8

DISCIPLINE Structural/H & H INSPECTOR T. Bennedum

DISCIPLINE Geotechnical INSPECTOR S. J. Poulos

AREA EVALUATED	CONDITION
OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a. Approach Channel	
General Condition	Fair to good.
Loose Rock Overhanging Channel	None.
Trees Overhanging Channel	Trees to 8' high growing on a bar upstream from spillway.
Floor of Approach Channel	Sand, boulders, some logs.
b. Weir and Training Walls	Stone masonry portion of training walls poor. Concrete weir good. 8 minor hairline cracks in 100' weir. 3 cracks thru 50' weir, at 18' from L, center & 16' from R.
General Condition of Concrete	
Rust or Staining	Rust on and under flashboard supports.
Spalling	Mortar cracked & weak. Some stones loose and fallen out.
Any Visible Reinforcing	None.
Any Seepage or Efflorescence	Minor efflorescence @ hairline cracks in weir & on mortar of TW's
Drain Holes	None
c. Discharge Channel	
General Condition	Fair.
Loose Rock Overhanging Channel	None.
Trees Overhanging Channel	Trees in and on both sides.
Floor of Channel	Bouldery.
Other Obstructions	None.

VISUAL INSPECTION CHECKLIST

9

DAM Sugar Hill Dam DATE November 7, 1979

DISCIPLINE Structural INSPECTOR T. Bennedum

DISCIPLINE No Geotechnical Features INSPECTOR -

AREA EVALUATED	CONDITION
OUTLET WORKS - SERVICE BRIDGE	Wooden walkway down D/S slope to outlet structure.
a. Super Structure	
Bearings	None.
Anchor Bolts	None.
Bridge Seat	None.
Longitudinal Members	Same as deck planks.
Underside of Deck	Appears sound.
Secondary Bracing	Only on 5 of 9 spans. Fair.
Deck	Planks sound, but very slippery.
Drainage system	None. Simply runs off.
Railings	Wooden posts set in rock riprap. Wobbly.
Expansion Joints	N/A
Paint	None.
b. Abutment & Piers	None. Wooden deck set on timber cross pieces which are set on rock riprap.
General Condition of Concrete	N/A
Alignment of Abutment	N/A
Approach to Bridge	Dirt path.
Condition of Seat & Backwall	N/A

APPENDIX B

ENGINEERING DATA

Section	Description
B1	Listing of Locations for Available Records and Data
B2	Drawings
B3	Copies of Past Inspection Reports and Data

APPENDIX B

SECTION B1

LISTING OF LOCATIONS FOR AVAILABLE RECORDS AND DATA

Owner: Central Vermont Public Service Corporation
77 Grove Street
Rutland, Vermont 05701
Attention: J. Douglas Graham,
Manager of Hydraulic Generation
(802) 773-2711

1) drawings
2) inspection reports
3) warning system

(Details and extent of data not known due to unwillingness of Owner to make such information available.)

Designer of Original Dam (1922): Vaughan Engineers, Boston, Mass.

(Believed to be no longer in business.)

Contractor for Original Dam (1922): Unknown

Designer of Raised Dam (1932): New England Public Service Corporation (NEPSCO)
(Location and business status unknown.)

Contractor for Raised Dam (1932): Sanders Engineering Company

(Location and business status unknown.)

Agency of Environmental Conservation
Department of Water Resources
Water Quality Division
Montpelier, Vermont 05602
Attention: A. Peter Barranco, Jr., P.E.
Dam Safety Engineer
(802) 828-2761

1) inspection reports
2) stage-area-storage data

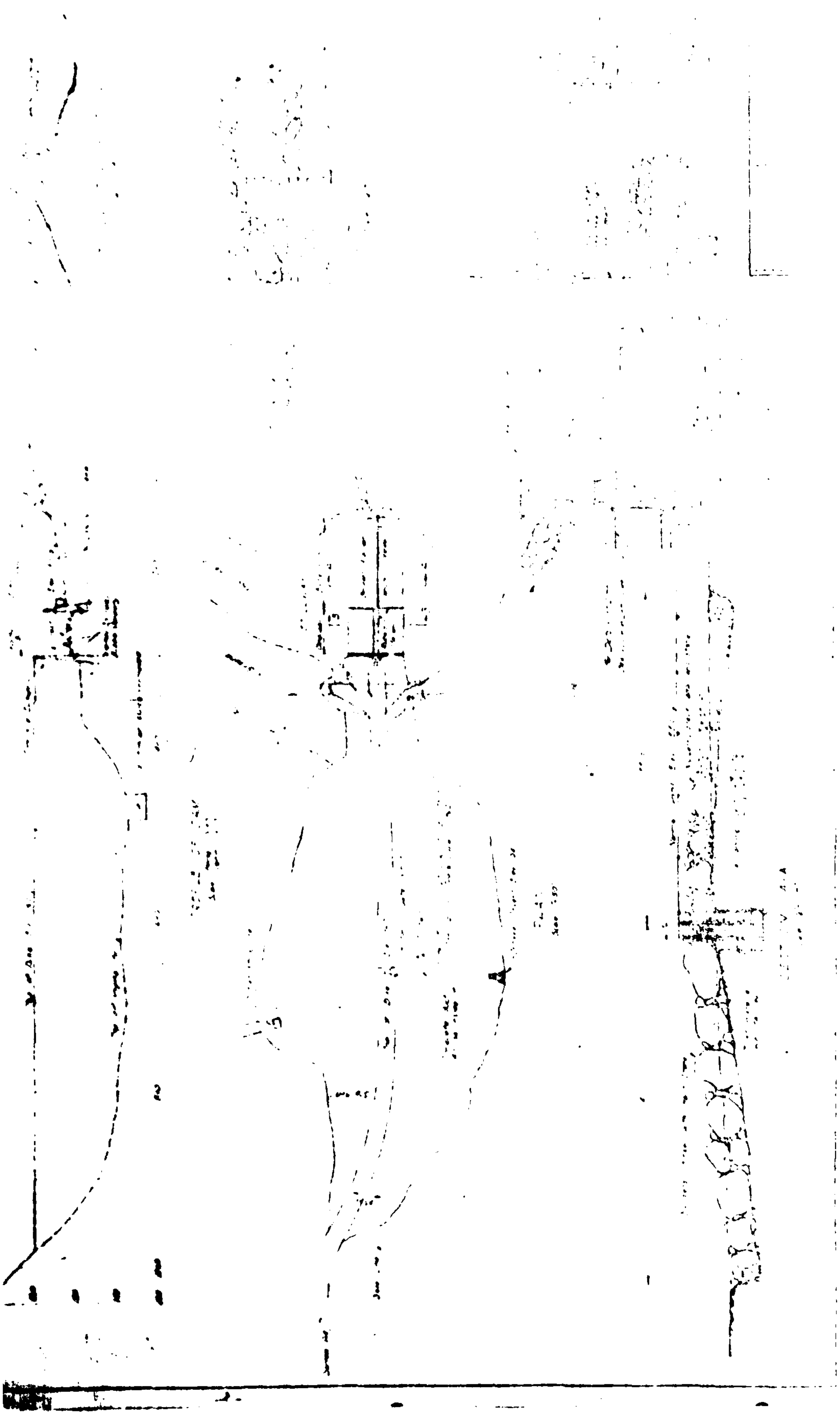
Vermont Public Service Board
State Office Building
120 State Street
Montpelier, Vermont 05602
Attention: Wayne Foster, Utility Engineer
(802) 828-2326

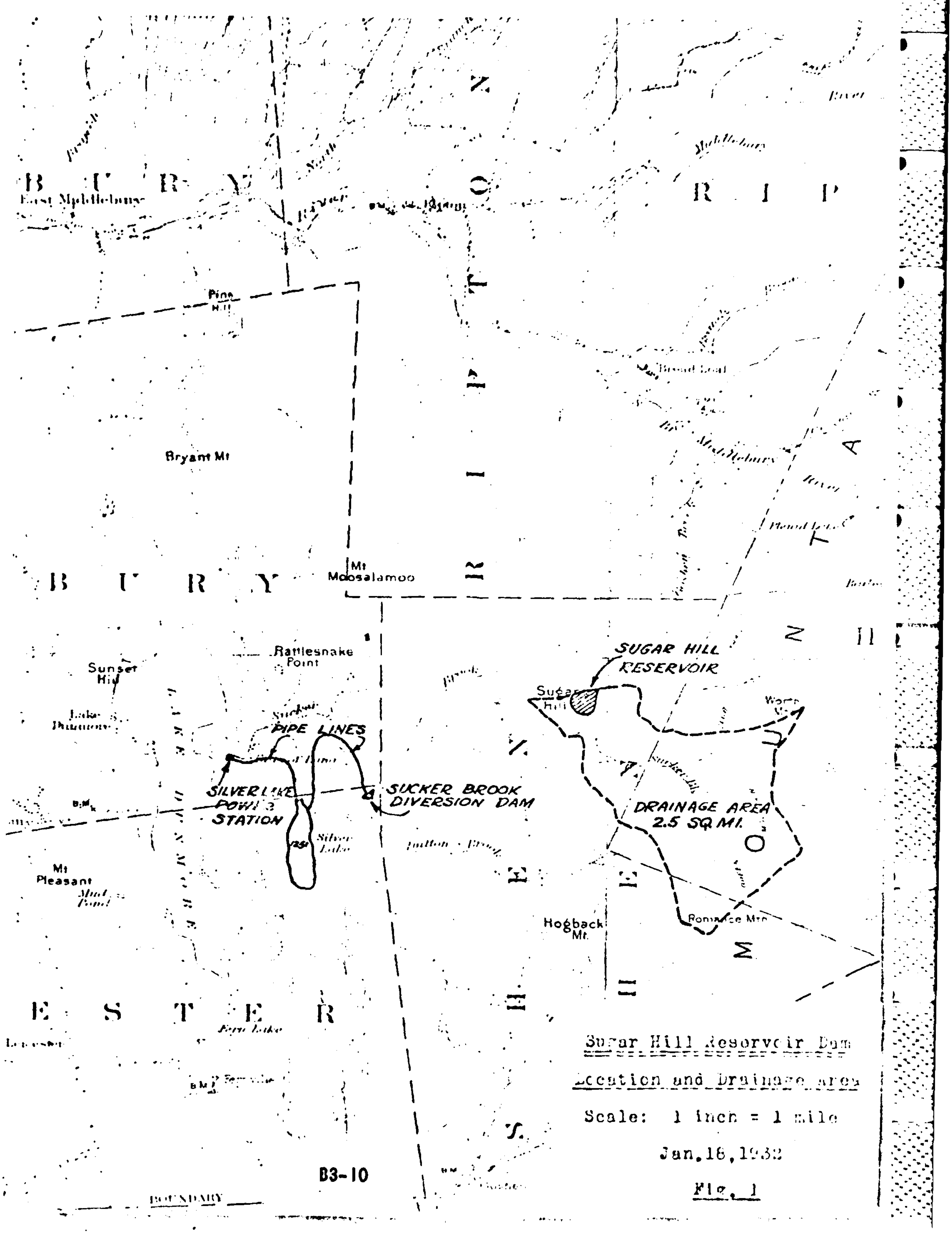
1) case numbers and old drawing

Vermont Public Records
133 State Street
Montpelier, Vermont 05602
(802) 828-3280

1) dam report and PSC approval order
2) correspondence

Concrete Conduit





Sugar Hill Reservoir Dam

Location and Drainage area

Scale: 1 inch = 1 mile

Jan. 18, 1932

Fig. 1

B3-10

Acknowledgments are made to the engineers of the Power Company for the full information freely supplied by them and other assistance and courtesies rendered. Also for information supplied by Mr. J. F. Vaughan, Engineer, of Boston relative to old dam.

Respectfully submitted,

H. [signature]

Accompanied by plans:-

Fig.1 (General Drainage Area Plan).

Sheet 392-3 of Power Co.

In the Office.

I obtained at my request, from Mr. F.H.Mason, Chief Engineer of the Power Company at Augusta, Maine, very full details of the old and new dams, their manner of construction and other pertinent details of the work, which have served as a basis for this report.

Other office work has included the checking of drainage area, reservoir capacity and adequacy of spillway. At the time of the 1927 flood the maximum discharge from Sugar Hill Reservoir was about 500 sec.ft. The spillway as lengthened will discharge this amount with a depth of about 1 foot over the crest. The spillway is ample in capacity.

CONCLUSIONS AND RECOMMENDATIONS

The old dam at Sugar Hill Reservoir was designed and supervised by competent engineers and apparently well constructed upon suitable foundations.

The dam as raised in 1931 was also under good engineering supervision and built according to proper plans. Apparently from the information available it was well built and properly supervised during construction.

The complete work as constructed in my judgment provides adequately for the public safety and its manner of construction is satisfactory.

The entire conduit and flume are now apparently in good condition.

Details of the various portions of the dam and its accessories - particularly portions added in 1951 - are shown upon Plan 392-3 of the Power Company appended.

INVESTIGATIONS

In the Field.

Before receiving your order of December 12, 1951 all the new construction work had been completed, so that no supervision during construction was possible.

Dec. 29, 1951 I visited the dam in company with Mr. R. A. Burdett of the Rutland office of the Power Company and made a careful inspection of the dam in its completed form - including inspection of materials in the borrow pits used for the embankment, which were satisfactory. Water level in the reservoir at this time was at approximately El. 125 or some 25 ft. above the outlet conduit level.

The earth embankment appeared to be well made and all visible concrete in the new spillway and outlet works was in satisfactory condition. The old 50 ft. length of spillway was also in good condition.

I received a generally good impression of the work and the manner in which it was done.

at the top of the slope, is normally open. This when necessary can be lowered by its own weight. On the reservoir side of the gate are 2" x 6" wooden racks about 18 ft. long flaring out to a width of 14 ft., laid on and at about the level of the 1 on 2 slope of the embankment.

At the lower end of the conduit is an 8" timber bulkhead in which are placed 2 - 6 inch, 2 - 10 inch and 6 smaller gate valves for accurate control of outlet water, which passes through a concrete plank lined flume about [illegible] ft. long x 6 ft. wide with two plank baffle weirs, one at El. 110 near the gate bulkhead and another, at about El. 104, [illegible] ft. downstream from the bulkhead.

The outlet conduit was evidently built with care, through firm material with proper cutoffs. It was inspected on November 24, 1931 for its full length by the inspector and found in good condition, with no signs of leakage or settlement.

The flume at the lower end was repaired by adding reinforced concrete to the walls and floor, with new planking. All concrete aggregate for this work, as well as for the new spillway, came from Mack's pit at Salisbury, Vt., which has been proven to be of good quality by use on various jobs.

Two eleven-hour shifts daily were employed, thus providing nearly continuous placing and rolling and resulting in the practical elimination of any freezing of earth during the work. Total embankment placed was about 35,000 cu.yds.

Work was done on a fee basis by the Sanders Engineering Co., a competent organization with much experience in such construction. It was also supervised by an inspector of the Power Company, with several visits as well by their engineer in charge.

Spillway. The old 50 ft. rubble stone masonry spillway at El. 155 was lengthened 1[illegible] ft. by a low reinforced concrete spillway 18" wide with crest at the same elevation (155), flanked downstream for about 30 ft. by large boulders and rock fill and upstream by boulders and gravel fill about the same distance. The spillway wall was about 8 ft. high, entering firm material for about 4 ft.

Outlet Conduit. This is a reinforced concrete 4 x 4 ft. in cross-section with 12" walls, roof and floor and center at about El. 101.5. It was built for the entire length necessary in [illegible].

An inclined emergency reinforced concrete gate 4 ft. wide by 5 ft. high, at angle of 60° with the horizontal, with chain control running to a [illegible]

masonry spillway at El. 155 an added amount of 100 ft., making the total spillway length 150 ft.; also repairs to the outlet portion of the concrete conduit through the dam and raising of the gate control mechanism on top of the dam at this outlet conduit.

Earth Embankment. In starting the new fill upon the old, bushes were grubbed off and the whole area ploughed and harrowed. On the original ground surface beyond the limits of the original embankment a central cutoff trench about 15 ft. wide and 5 ft. deep was dug with power shovel and the remaining area ploughed and grubbed.

The top width of the embankment was made 20 ft. at El. 160 and both side slopes 1 on 2. This raised the dam about 30 ft., making its total height about 60 ft. Two borrow pits near the dam were used, one for fine material and the other for courser material. Fine material was placed in the center and most of the upstream section, with coarser material at the upstream slope and the downstream bank. The upstream slope was finished with loose rock fill. Material for the embankment was hauled by tractors, spread by bulldozers in 6 inch layers and compacted by continuous travel of this equipment, with some added compacting during temporary cessation of filling.

The drainage area of Sugar Hill Reservoir is about 2.5 sq.miles; it has a water area at new spillway level, El. 155, of about 75 acres and a storage capacity of about 70 million cu.ft.

<u>Old Dam.</u>

The old dam was constructed during the season of 1922, of rolled earth fill about 500 ft. long and about 30 ft. in maximum height. The old earth dam was apparently well built, of suitable materials and under the careful engineering supervision of Mr. J. H. Vaughan of Boston. It has apparently shown no appreciable leakage during its use of some 8 or 9 years time. A rubble masonry spillway 50 ft. long at El. 155 was also constructed at this time, as well as a 4' x 4' concrete outlet conduit underneath the dam. A temporary wooden flume spillway 28 ft. wide was left in the dam at the end of the season, with the intention of building the dam up to about El. 160 during the next season. No further work was done, however, until late in 1931.

<u>Dam as Raised.</u> (See Sheet 392-3 appended.)

During the late fall of 1931 plans were made to raise the dam to approximately the level originally contemplated. Work was begun on the [illegible] embankment on November 15, 1931 and this was completed on December 5, 1931. The next work [illegible]

H. K. BARROWS
CONSULTING ENGINEER
6 BEACON STREET
BOSTON

RICHARD S. HOLMGREN
PRINCIPAL ASST. ENGINEER

January 18, 1932

Hon. Henry B. Shaw, Chairman,
Public Service Commission,
Montpelier, Vt.

No.1697 - Sugar Hill Reservoir Dam.

Dear Sir:-

In accordance with the order of your Commission dated December 17, 1931, I submit the following report upon the dam at Sugar Hill Reservoir near Goshen, Vermont, reconstructed by raising its level about 30 ft. during November-December 1931.

DESCRIPTION

Sugar Hill Reservoir Dam of the Central Vermont Public Service Corporation is located in the town of Goshen upon Sucker Brook, which is a tributary to Lake Dunmore entering the lake from the east. Lake Dunmore is in turn tributary to Leicester River, which flows into Otter Creek near Leicester Jc. - about 8 miles above Middlebury. Storage from Sugar Hill Reservoir is used under a head of about 670 ft. in the Silver Lake development of the Company, the power station for which is located upon Sucker Brook a short distance east of its entrance to Lake Dunmore.

[illegible]

Submitted to Hon. [illegible]

Vermont Public [illegible]

[illegible]

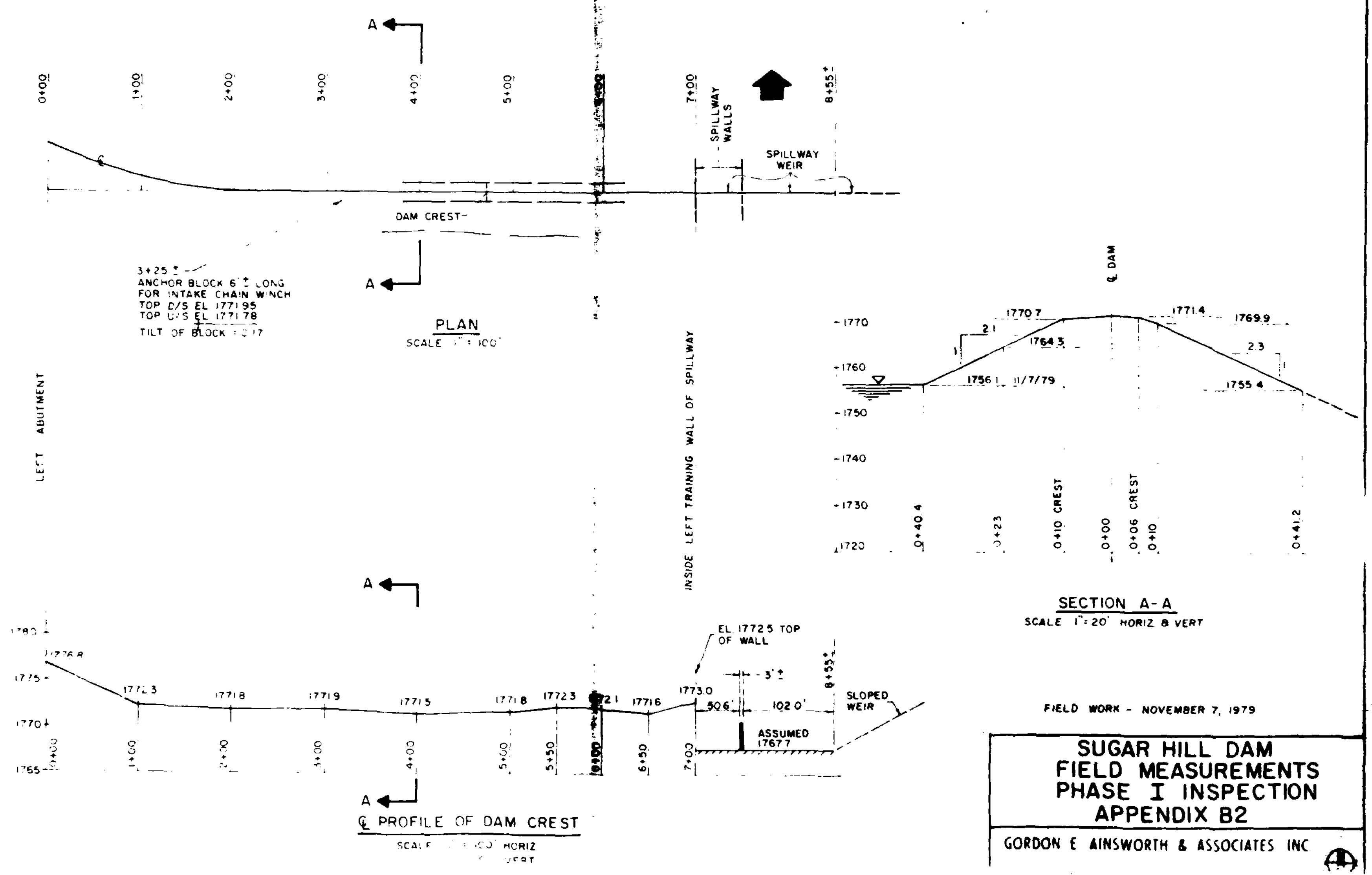
By H. E. [illegible]

APPENDIX B

SECTION B3

COPIES OF PAST INSPECTION REPORTS AND DATA

TABLE OF CONTENTS



SUGAR HILL DAM
FIELD MEASUREMENTS
PHASE I INSPECTION
APPENDIX B2

GORDON E AINSWORTH & ASSOCIATES INC

No. 1697

Petition of Central Vermont Public Service Corporation for approval of plans and specifications to increase size of dam at Sugar Hill Reservoir in Goshen.

ORDER

The petition, filed December 9, 1951, in substance alleges that petitioner, a Vermont corporation authorized and engaged in the generation, manufacture and sale of electricity, is the owner of an artificial storage reservoir located in Goshen in Addison County on Sucker Brook known as "Sugar Hill Reservoir"; that the dam impounding the water of said reservoir was planned and partially built in 1922 and 1923 by Hortonia Power Company.

The original plan of said dam contemplated an impervious earth-filled dam sixty (60) feet in height, riprapped on the upper slope thereof, having a width at the top of said fill of fifteen (15) feet and with a rubble masonry spillway fifty (50) feet in length at fifty-five (55) foot level, and with an emergency gate at inlet of discharge tunnel or tube with an outlet control consisting of rectangular tunnel or tube four (4) feet square, from which water was ultimately discharged through a cyclindrical tube four (4) feet six (6) inches in diameter, controlled at the outlet thereof by graduated valves into a rectangular concrete stilling pool and thence into the stream from which, by diversion works, it was conveyed through a penstock to Silver Lake, so-called, in the Town of Leicester.

Before construction was completed and on November 1st, 1923, construction was discontinued, leaving an earth-filled dam or dyke at a minimum level of about thirty-eight (38) feet, and of approximately fifteen (15) feet width at the top, and with a temporary timber spillway at thirty-five (35) foot level.

That to the end that said storage reservoir might be completed and made adequate for the purposes of the petitioner in the conduct of its business, and on or about the 5th day of November, 1951, revised plans for

the completion of said reservoir were prepared, which plans called for the completion of said reservoir to the height originally contemplated, namely, sixty (60) feet, by increasing the thickness of said dyke or fill so that its width at the top thereof would be twenty (20) feet instead of fifteen (15) feet, and the maximum width at the base approximately three hundred thirty (330) feet, and providing in place of the spillway originally contemplated of fifty (50) feet in length, a spillway one hundred fifty (150) feet in length at said fifty-five (55) foot level, being one hundred (100) feet length additional to the plan originally contemplated, said one hundred (100) feet additional of spillway to have a concrete core wall with rubble paving on both upstream and downstream slopes; also to provide for general repairs and reinforcement of the discharge outlet and stilling pool.

When the work so contemplated should be completed, said reservoir at spillway level would overflow seventy-four and one-tenth (74 1/10) acres, and at the level at the top of the dam would overflow eighty-two and four-tenths (82 4/10) acres.

The capacity of said completed storage reservoir at spillway level would be approximately 69,646,000 cubic feet. The drainage area embraced within and above said dam is two and seventy-five hundredths (2.75) square miles.

The work of completion to be performed by Sanders Engineering Company, under the engineering direction of F. W. Harris, civil engineer for New England Public Service Corporation; and to the end that said construction work might be completed while weather conditions permitted, the same was begun on or about November 10, 1931, and has been substantially completed in accordance with the plans hereinbefore outlined and referred to.

In order that no question may arise with respect to the completion of said storage reservoir by reason of the provisions of No. 80 of the Acts of 1929, entitled "An Act Relating to the Construction and Repair of Dams, and to Repeal Sections 4388, 4889 and 4390 of the General Laws," and that the petitioner may in all respects conform to and abide by the laws of Vermont, which may in any way or manner relate to or affect the completion

of construction of said dam, the petitioner represents that it has given notice of this petition to the Selectmen of said Town of Goshen, in the County of Addison aforesaid, in which said dam is located, and has delivered to said Selectmen a copy thereof.

The petition concludes with a prayer that this Commission review the plans and specifications attached to petition and make such additional investigation respecting said dam as it deems necessary in accordance with No. 30 of the Acts of 1929.

The Commission designated, with the approval of the Governor, H. K. Barrows of Boston, Massachusetts as the engineer to investigate the property, review the plans and specifications and make such additional investigations as the Commission might deem necessary.

On January 20, 1932 the written and signed report of the engineer was filed. This report is to the effect that

The old dam at Sugar Hill Reservoir was designed and supervised by competent engineers and apparently well constructed upon suitable foundations.

The dam as raised in 1931 was also under good engineering supervision and built according to proper plans. Apparently from the information available it was well built and properly supervised during construction.

The complete work as constructed in the judgment of the engineer provides adequately for the public safety and its manner of construction is satisfactory.

We, therefore, approve the plans and specifications and the construction of the dam of the petitioner above described in the town of Goshen.

Dated at Montpelier, County of Washington, State of Vermont, this 21st day of January, A. D. 1932.

Henry B. Ear[illegible])
Stephen S. Cushing)

Public Service Commission of Vermont

OFFICE OF CLERK
Filed: January 21, 1932
Attest: [illegible]

A-1

REPORT ON SUGAR HILL DAM

IN GOSHEN, VERMONT

This report on Sugar Hill dam follows an examination and study of the structure.

Pertinent data

1.	Owner & operator of dam	-	Central Vermont Public Service Corporation
2.	Purpose of dam	-	Storage for Silver Lake hydro development
3.	Stream location	-	Sucker Brook
4.	Town location	-	Goshen, Vermont
5.	Reservoir surface area	-	75 acres
6.	Reservoir volume	-	70,000,000 cu. ft.
7.	Drainage area	-	2.5 sq. mi.

Historical brief

Construction of this dam was begun in 1922 by the Hortonia Power Company, previous owner. An earth dam 60 ft. high was contemplated but the project was abandoned after the earth fill reached a height of about 30 ft.

The Central Vermont Public Service Corporation purchased the rights and completed the dam in 1931-32. The project was approved by PSC under Case #1697. H. K. Barrows, Consulting Engineer, was retained as the Commission's engineer in the matter.

Description of dam

The layout and dimensions of the existing dam are shown in the attached NEPSCO drawing #392-3. Other details are contained

in the case file.

In general, the structure is an earth embankment about 700 ft. in total length and 60 ft. in maximum height. It has a top width of 20 ft. Both faces slope at about 1 on 2. A stone-fill riprap cover protects the upstream face and also the downstream toe.

The spillway is located at the west end of the embankment. It consists of an old rubble masonry weir 50 ft. long plus an added 100 ft. concrete weir, all on an earth foundation. Its crest is 5 ft. below the top of the dam. The channel for about 30 ft. on both the upstream and downstream side of the weir is paved with large boulders and with rock and gravel fill.

An outlet structure is provided at the maximum section through the center of the dam. It consists of a 4 ft. square reinforced concrete conduit about 300 ft. long. A trash rack and emergency gate are provided at the upstream end. A timber bulkhead at the downstream end contains a number of small gate valves for close control of discharge.

Observed condition of the dam

As observed by the writer the condition of the dam was noted as follows:

Spillway - Some seepage through or under the cutoff wall. However, there was no apparent effect on its stability. Spillway clear - flashboard supports in place but no boards are used. Discharge channel in earth recently cleared and reinforced with small check dams.

A-1

INSPECTION REPORT
ON
Goshen (Sugar Hill) Dam

1. Date of inspection May 13, 1953 2. Water conditions Almost crest level

GENERAL DATA:

3. Location of dam Sucker Brook, town of Goshen

4. Owner and operator Central Vermont Public Service Corp.

5. Characteristic features of dam Embankment 60 ft. high on earth foundation

6. Other related data (Contained in writer's initial report on structure)

OBSERVATIONS:

7. Condition of structure See reverse side

8. Condition of equipment In operating order

9. Operation Satisfactory

10. Maintenance Good

REMARKS:

Dam is in acceptable condition.

This inspection made with R. L. Gouchoe, company engineer.

B3-18 Inspected by Stephen H. Haybrook

7. <u>Embankment</u> - The one uncomplimentary comment is in regard to the seepage condition. An apparent boil (water emerging from the gound with force) was discovered below the downstream toe at maximum section. Although of little significance at present it is to be kept under observation for any transportation of material and quantity of flow.

<u>Spillway</u> - Overgrowth is stabilizing the loose material in discharge channel. Tree growth at crest to be cleared.

<u>Outlet Structure</u>- Minor deteriation at discharge end.

REPORT OF INSPECTION OF DAM

Reference: Public Service Commission letter of May 25, 1954.

JUL 2 8 1954

1. NAME AND LOCATION OF DAM

Sugar Hill Dam - Goshen, Vermont

P.S.C. Report 1952 - Class 2

2. BY WHOM INSPECTION WAS MADE

Byron O. McCoy, Chas. T. Main, Inc., Boston, Mass.

R. Reid, Superintendent of Stations, Central Vermont Public Service Corp.

R. L. Gouchoe, General Engineer, " " " " "

3. DATE OF INSPECTION

June 10, 1954

4. RESULT OF INSPECTION

Little seepage at downstream toe of the embankment and no sign of a "boil" referred to in previous year State inspection. The pond level was 13 inches below the spillway at the time of this inspection. The "boil" was observed several times during 1953 and it did not increase as the pond filled, but rather dropped off as the ground water table in the general area.

Inspection indicates that the dam does not have any defect which endangers life and property and will withstand without disaster flash floods which may reasonably be expected to occur.

CENTRAL VERMONT PUBLIC SERVICE CORPORATION

Signed Harold J. Pein
Executive Vice President

RLG:RS
6/28/54

Elev.	Sq. Ft.	Acres	Cu. Ft. Storage	KWH
117	450,000	10.3	2,221,000	23,800
118	515,000	11.8	2,753,000	29,500
119	580,000	13.3	3,2[illegible]1,000	30,400
120	644,000	14.8	3,713,000	44,300
121	700,000	16.1	4,391,000	50,720
122	756,000	17.4	5,[illegible],000	60,210
123	811,000	18.7	6,068,000	69,600
124	867,000	20.0	6,851,000	79,100
125	922,000	21.2	7,635,000	88,5[illegible]
126	1,001,000	23.0	8,554,000	102,000
127	1,080,000	24.8	9,823,000	115,500
128	1,158,000	26.6	10,922,000	128,000
129	1,237,000	28.4	12,111,000	142,400
130	1,315,000	30.2	13,230,000	155,900

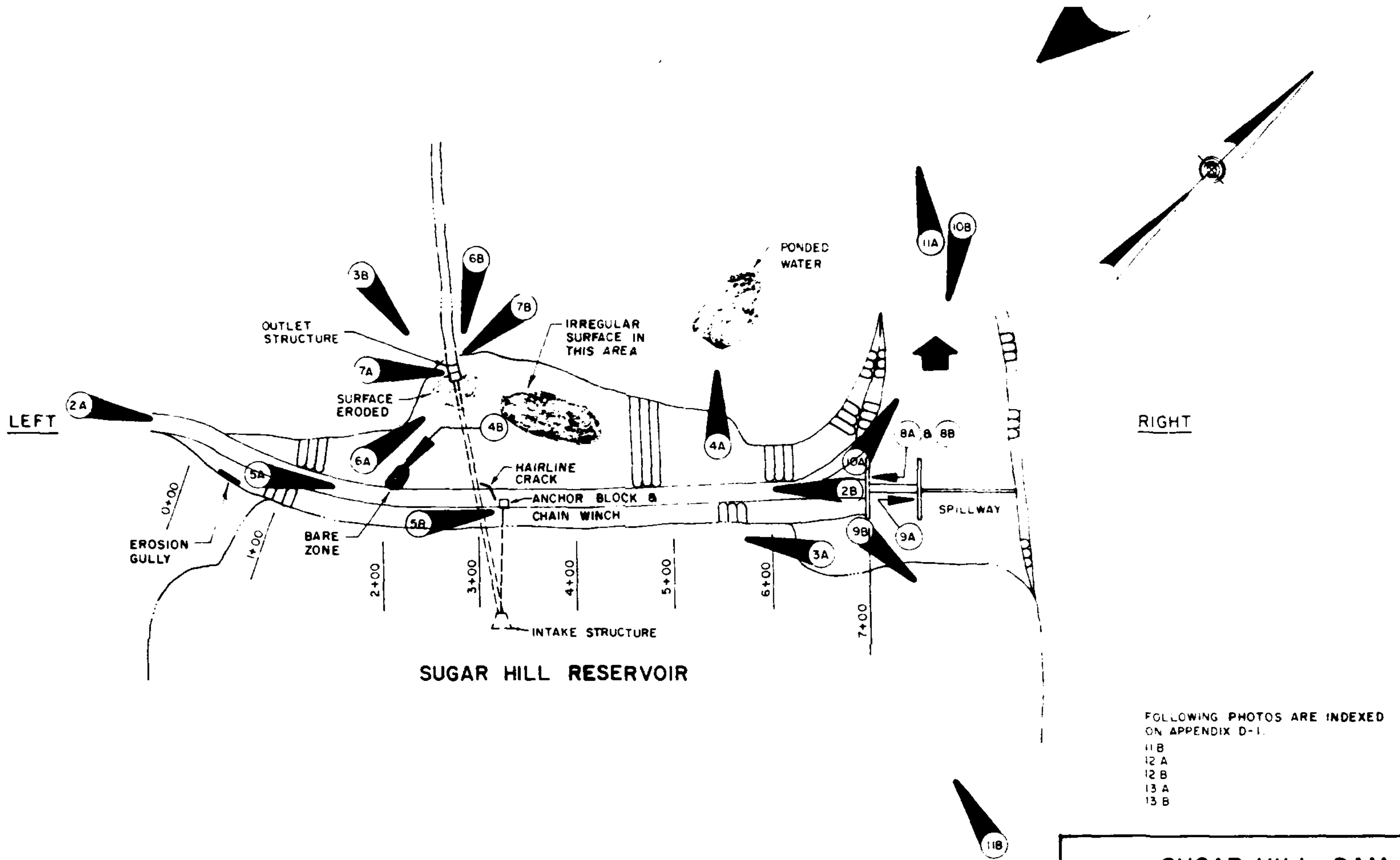
Elev.	Sq. Ft.	Acres	Cu. Ft. Storage	KWH
134	1,574,000	36.2	19,358,000	[illegible]
135	1,639,000	37.6	20,615,000	227.5
136	1,707,000	39.2	22,283,000	250.[illegible]
137	1,774,000	40.7	25,231,000	283.[illegible]
138	1,852,000	42.4	26,932,000	31[illegible]
139	1,908,000	43.8	27,837,000	33[illegible]
140	1,977,000	45.4	28,655,000	[illegible]
141	2,083,000	47.8	31,025,500	[illegible].7
142	2,169,000	50.2	35,135,000	[illegible]
143	2,324,000	51.6	36,375,000	[illegible]
144	2,399,000	55.0	38,615,000	[illegible]
145	2,505,000	57.5	40,855,000	50[illegible]
146	2,583,000	59.3	43,553,000	[illegible].3
147	2,661,000	61.1	46,250,000	565.5
148	2,738,000	62.9	48,940,000	[illegible]
149	2,816,000	64.7	51,645,000	[illegible]
150	2,894,000	66.5	54,343,000	[illegible]
151	2,961,000	68.0	57,404,000	[illegible]
152	3,021,000	69.5	60,465,000	[illegible]
153	3,090,000	71.0	63,515,000	773.[illegible]
154	3,160,000	72.5	66,595,000	810.[illegible]
155	3,227,000	74.1	69,626,000	847.7
156	3,299,000	75.8	73,058,000	[illegible]
157	3,372,000	77.4	76,162,000	[illegible]
158	3,444,000	79.1	79,870,000	[illegible].9
159	3,512,000	80.7	83,278,000	1011.2
160	3,583,000	82.4	86,685,000	1053.0

NOTE

MINIMUM ELEV. TO WHICH WATER CAN BE DRAWN [illegible]
BOTTOM OF RACKS = ELEV. 106
MINIMUM ELEV. TO WHICH WATER SHOULD BE
DRAWN FOR ORDINARY OPERATING IS ELEV. [illegible]
K.W.H. AT SILVER LAKE STATION ARE FIGU[illegible]
TO THIS ELEV. USING 83 CU.FT. PER K.W.H.
QUANTITIES ARE FIGURED AT THE FIVE FOOT
CONTOURS, OTHER CONTOUR QUANTITI[illegible]

APPENDIX C

PHOTOGRAPHS



SCALE NONE

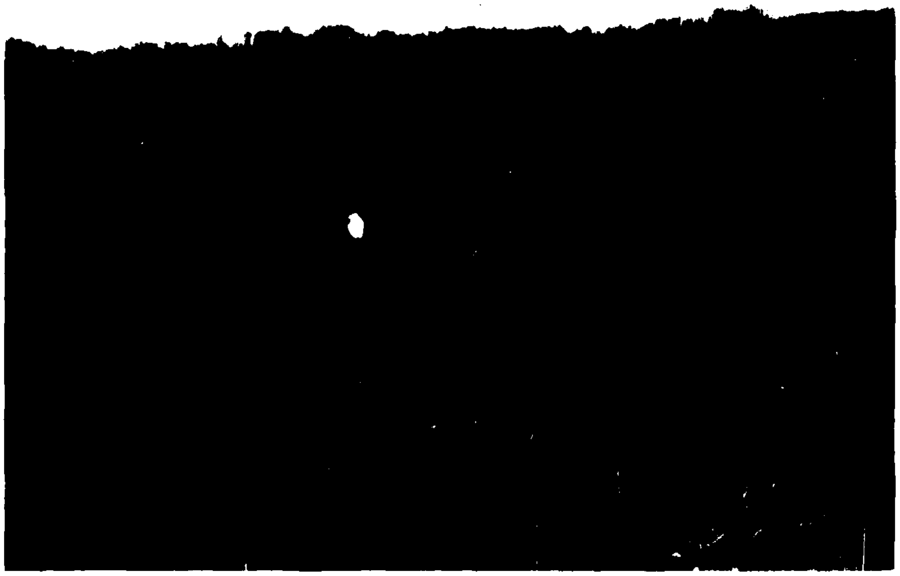
SUGAR HILL DAM
PHOTO INDEX MAP
APPENDIX C-1

GORDON E AINSWORTH & ASSOCIATES INC

20 SUGARLOAF ST SOUTH DEERFIELD MASS 01373

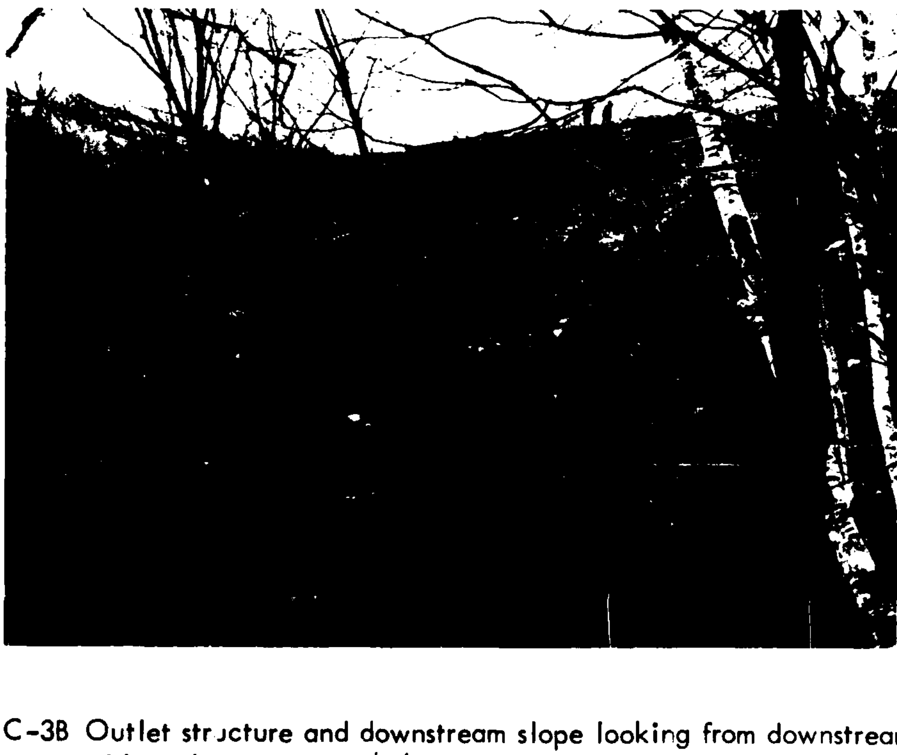
FEBRUARY 79 DWG NO 80-55

C-2A Dam crest looking from left abutment toward right abutment
11/7/79

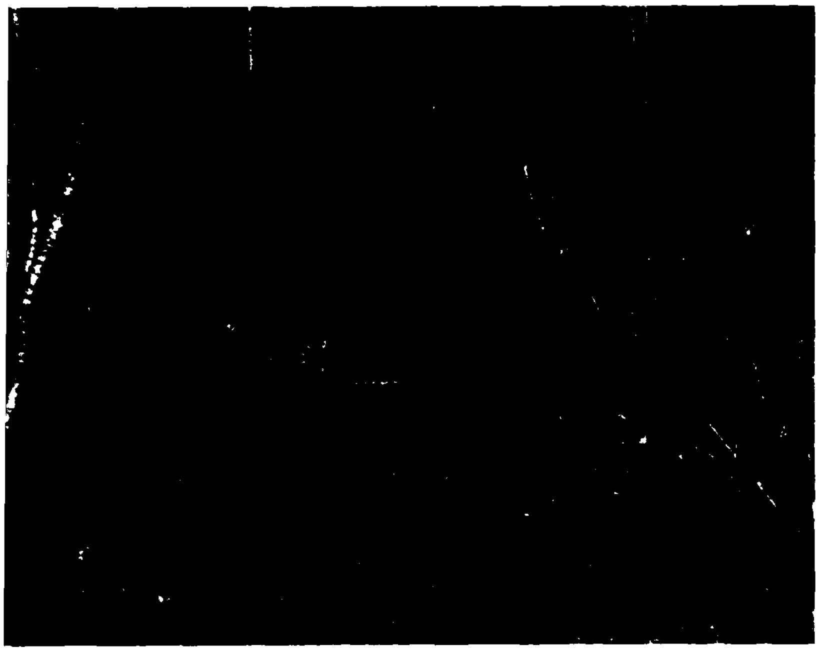


C-2B Dam crest looking from spillway toward left abutment - 11/7/79

C-3A Upstream slope looking from left of spillway - 11/7/79



C-3B Outlet structure and downstream slope looking from downstream of left abutment - 11/7/79

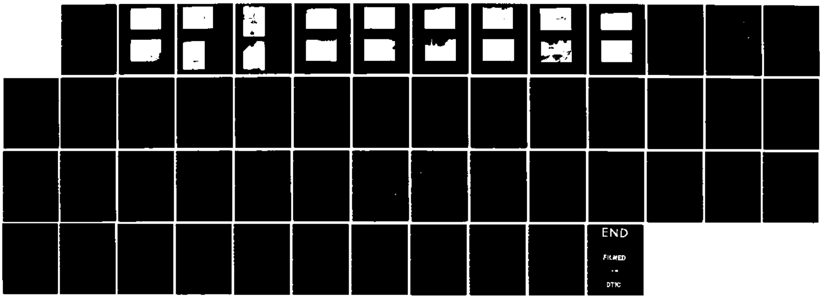


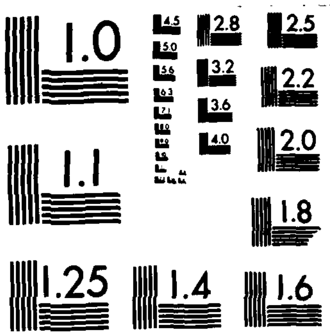
ı Ponded water at downstream toe of embankment - 11/7/79

C-4B Surface erosion in ıiprapped area of downstream slope - 11/7/79

AD-A156 023 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS SUGAR HILL DAM (VT 00..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV FEB 80 2/2

UNCLASSIFIED F/G 13/13 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

C-5A Crest of dam looking from Sta 1 + 50 $\pm$ toward right abutment. Note tilt of dam crest toward the downstream slope - 11/7/79

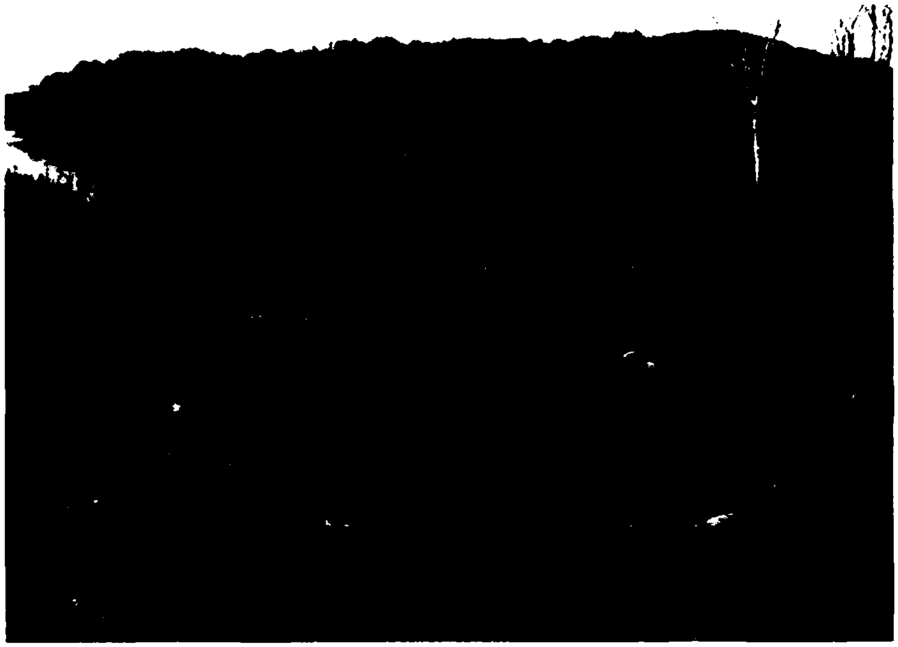
C-5B Anchor block and chain winch for emergency inlet gate 11/7/79

C-6A Service walkway to outlet structure - 11/7/79

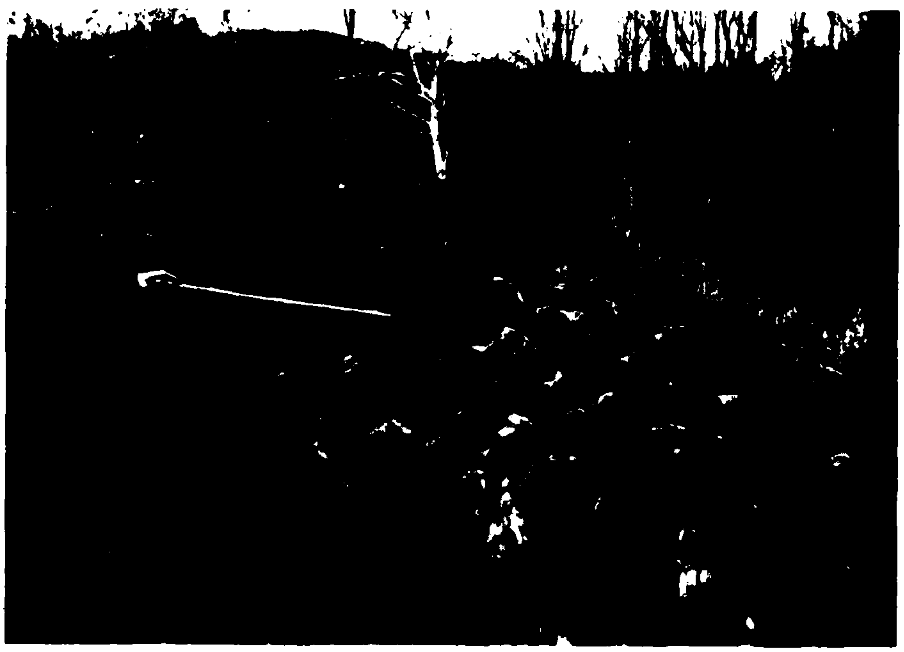
C-6B Outlet structure - 11/7/79

C-7A Crack in top section of outlet structure on left front corner 11/7/79

C-7B Eroded concrete and exposed steel reinforcement in outlet channel on left side under outlet structure 11/7/79



C-8A Spillway weir looking toward left training wall - 11/7/79



C-8B Left training wall of spillway. Note deterioration of stone masonry - 11/7/79

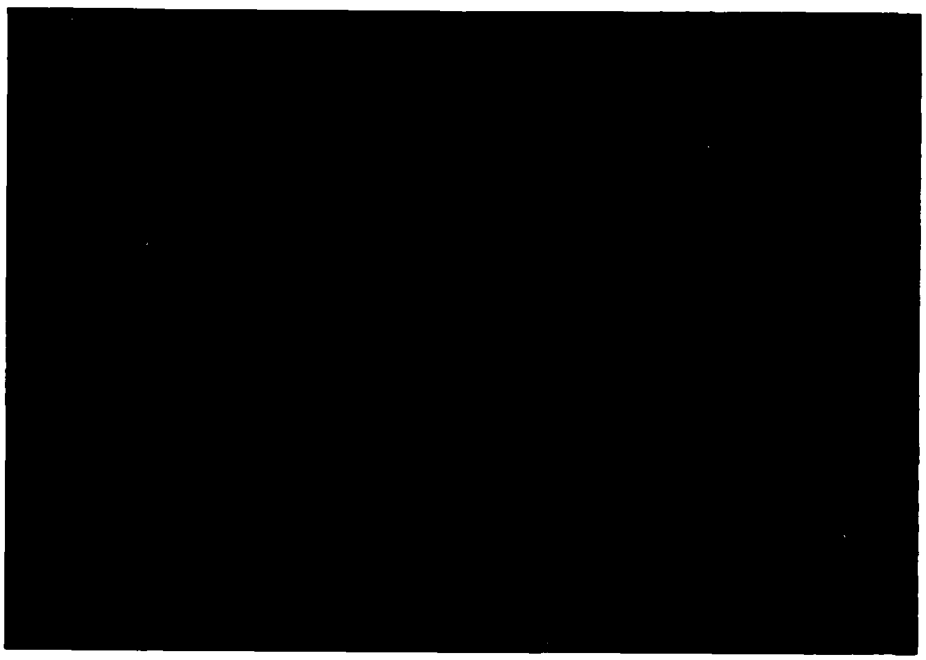
C-9A Spillway weir looking from left training wall toward right abutment - 11/7/79

C-9B Spillway approach channel looking from left training wall 11/7/79

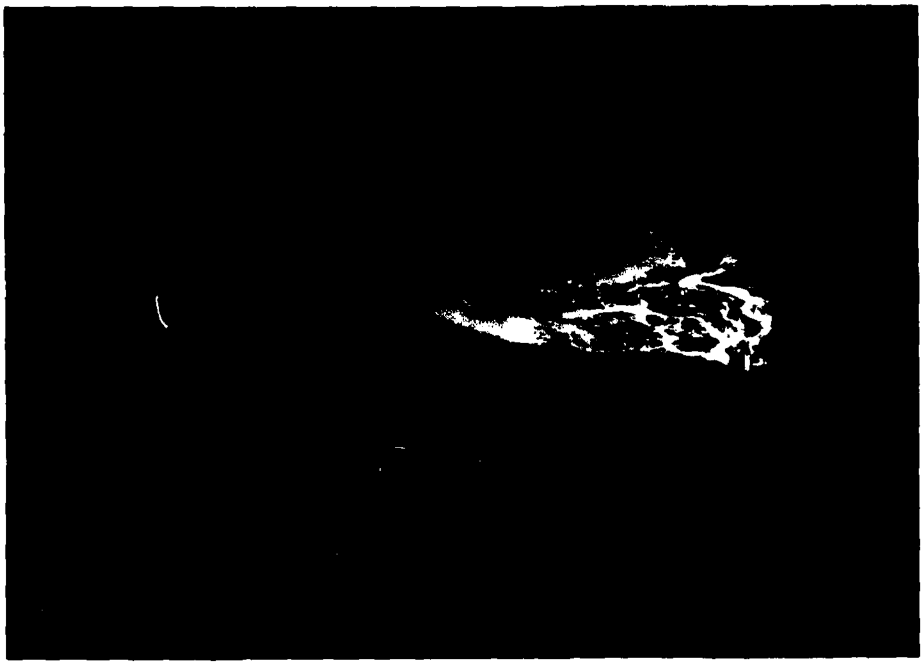
C-10A Spillway discharge channel looking downstream from left training wall - 11/7/79

C-10B Gabion structure in spillway discharge channel looking toward spillway - 11/7/79

C-11A Spillway discharge channel looking downstream from gabion structure - 11/7/79



C-11B Aerial overview of reservoir and dam looking downstream 11/30/79



C-12A Aerial overview of Sucker Brook Diversion Dam and Reservoir looking upstream - 11/30/79

C-12B Vermont State Route No. 53 bridge over Sucker Brook near Lake Dunmore. Note top of powerhouse for Silver Lake Hydroelectric Development visible over left end of bridge - 11/8/79

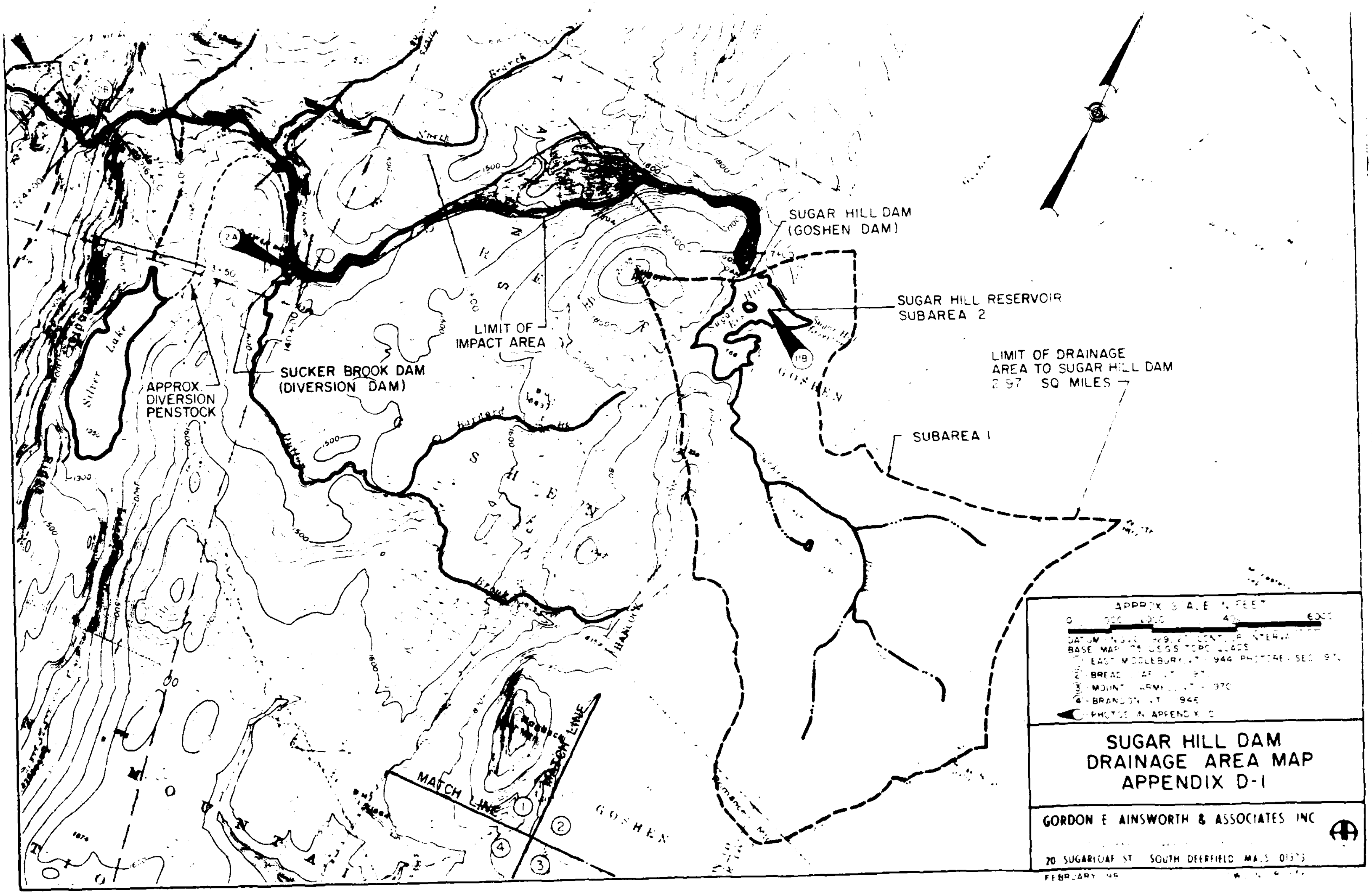
C-13A Aerial overview of downstream hazard area along Lake Dunmore. Sugar Hill Dam is in the mountains in the background - 11/30/79

C-13B Aerial overview of downstream hazard area along Lake Dunmore. Note Vermont State Route No. 53 across center, Branbury State Park in left center and outlet to Sucker Brook in right foreground - 11/30/79

APPENDIX D

HYDRAULIC AND HYDROLOGIC COMPUTATIONS

TABLE OF CONTENTS



SUGAR HILL DAM
DRAINAGE AREA MAP
APPENDIX D-1

GORDON E AINSWORTH & ASSOCIATES INC

70 SUGARLOAF ST SOUTH DEERFIELD MASS 01373

G. E. Ainsworth Associates
20 Sugarloaf Street
S. DEERFIELD, MA 01373
Phone 665-2161

JOB SUGAR HILL DAM
SHEET NO. ______ OF ______
CALCULATED BY ELV DATE 1/25/80
CHECKED BY JPB DATE 2/80
SCALE 21-06-79106

ELEVATION - AREA - STORAGE COMPUTATIONS

SUGAR HILL
RESERVOIR VOLUME: COMPUTED BY PROGRAM USING METHOD OF CONIC SECTIONS: $\Delta V_{12} = \frac{h}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right)$

	ELEVATION (NGVD - ft.)*	AREA*** (acres)		V_T ** (acre-feet)
	1717	0		0
	1723	2.0		4
	1728	7.3		26
	1733	14.8		80
	1738	21.2		170
	1743	30.2		297
	1748	37.6		467
	1753	45.4		674
	1758	57.5		930
	1763	66.5		1240
LWAY ST →	1768	74.1	SAY 74	1591
	1769	75.8		1666
	1770	77.4		1743
	1771	79.1		1821
ST →	1771.5	79.9	EST. SAY 80	1861
	1772	80.7		1901
	1773	82.4		1983
	1780	102.7		2629

* APPLICATION REPORTS AND PLANS SUBMITTED IN 1930'S BY CENTRAL VERMONT PUBLIC SERVICE CORPORATION TO THE VERMONT PUBLIC SERVICE COMMISSION UTILIZE AN ARBITRARY DATUM. NGVD ELEVATIONS ARE 1,613' HIGHER THAN THIS ARBITRARY DATUM.

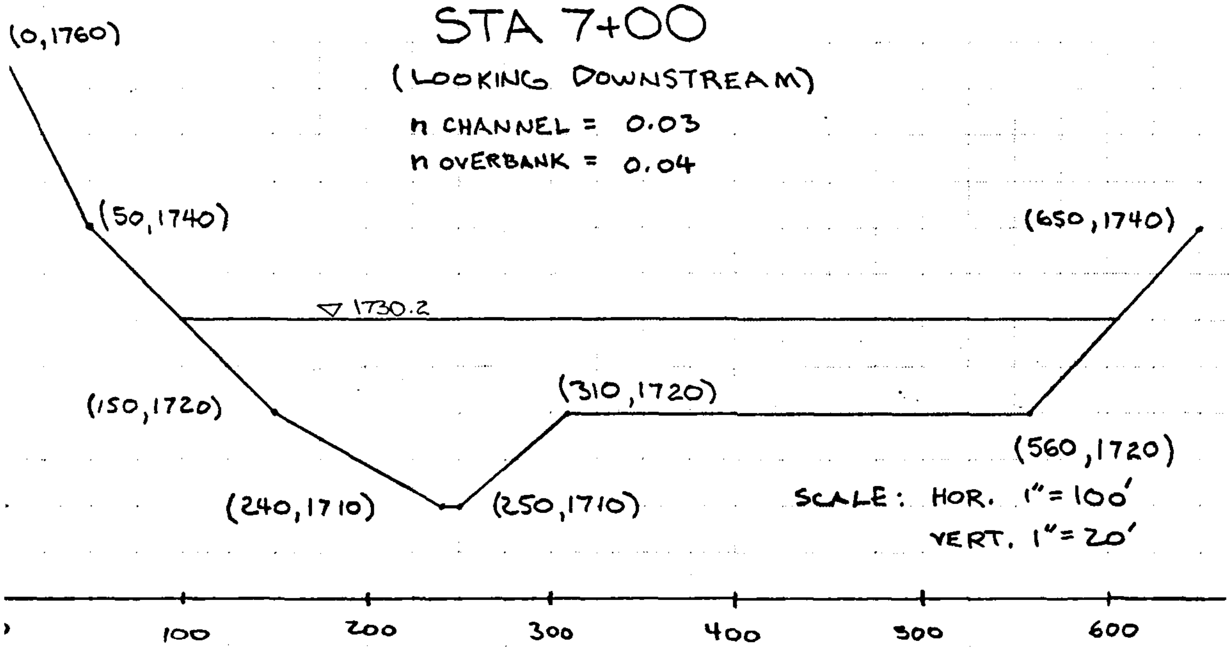
** COMPUTED BY HEC-1 DB COMPUTER PROGRAM.

*** ELEVATION - AREA DATA FROM FILES OF VERMONT DAM SAFETY ENGINEER & REPRODUCED AS APPENDIX B3-21

GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
[SOU]TH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

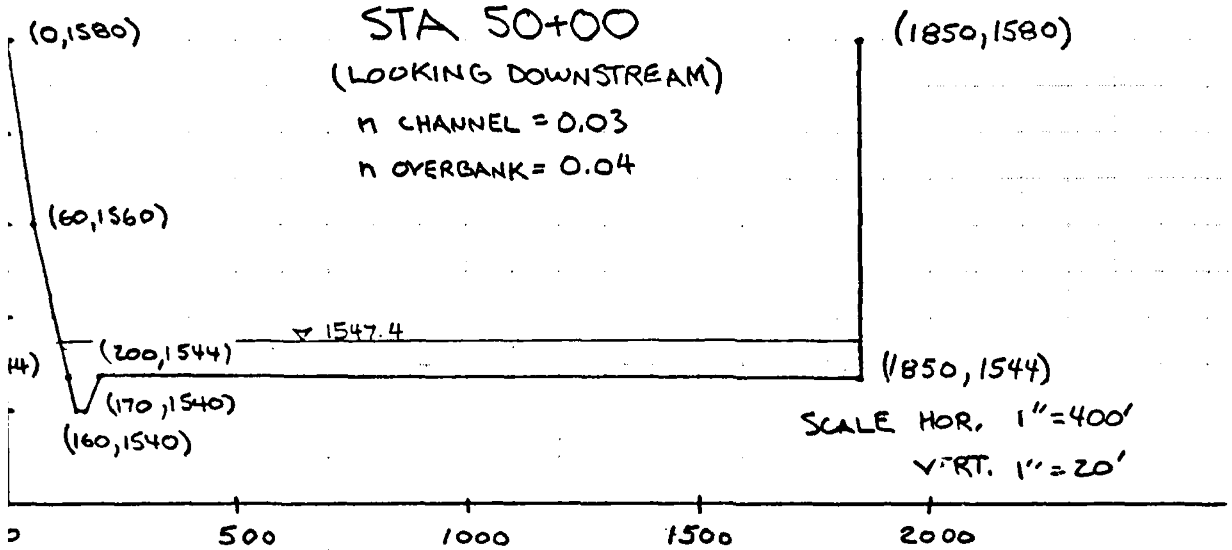
JOB SUGAR HILL DAM
SHEET NO. ______ OF ______
CALCULATED BY ELV DATE 2/1/80
CHECKED BY JPB DATE 2/80
SCALE 21-06-79106

STA 7+00
(LOOKING DOWNSTREAM)
n CHANNEL = 0.03
n OVERBANK = 0.04



FLOW AREA = $\frac{S}{L} = \frac{88}{700} \times 43560 = 5476$ SF

STA 50+00
(LOOKING DOWNSTREAM)
n CHANNEL = 0.03
n OVERBANK = 0.04



FLOW AREA = $\frac{S}{L} = \frac{587}{4300} \times 43560 = 5946$ SF

SUMMARY OF DAM SAFETY ANALYSIS

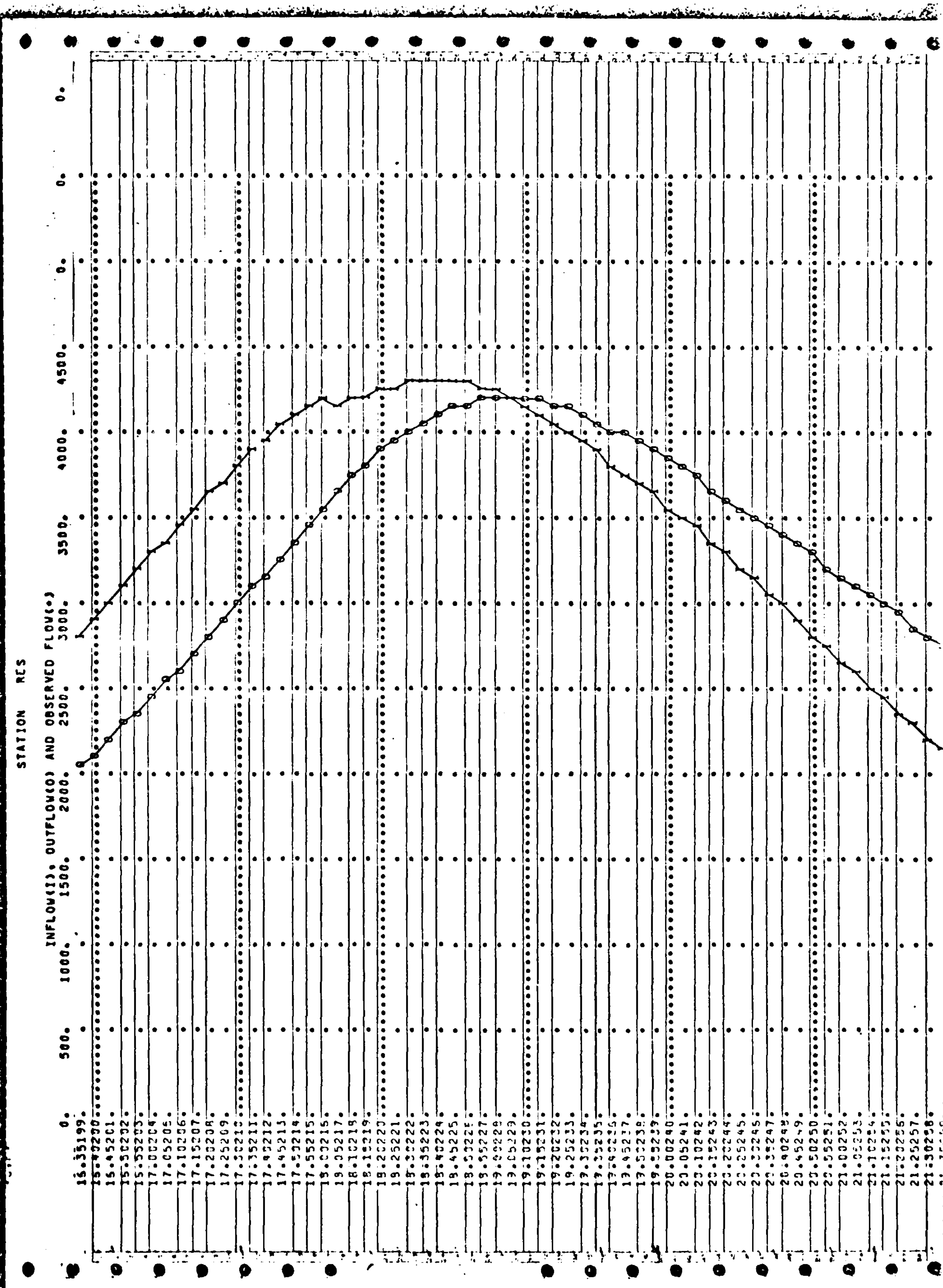
PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1768.00	1768.00	1771.50
STORAGE	1591.	1591.	1861.
OUTFLOW	0.	0.	3032.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	1772.14	0.64	1912.	4204.	3.67	19.08	0.00
0.50	1770.66	0.00	1794.	2009.	0.00	19.33	0.00

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

				RATIOS APPLIED TO FLOWS	
OPERATION	STATION	AREA	PLAN	RATIO 1 1.00	RATIO 2 0.50
HYDROGRAPH AT	SA-1	2.85 (7.39)	1	4291. (121.52)(	2146. 60.75)(
HYDROGRAPH AT	SA-RES	0.12 (0.30)	1	1485. (42.05)(	743. 21.03)(
2 COMBINED	SA-2C	2.97 (7.69)	1	4301. (121.79)(	2151. 60.90)(
ROUTED TO	RES	2.97 (7.69)	1	4204. (119.05)(	2009. 56.89)(


STATION RES
INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)
0.
500.
1000.
1500.
2000.
2500.
3000.
3500.
4000.
4500.
0.
0.
0.

```
     CAPACITY=       0.       4.      26.      80.     170.     297.     467.     674.     930.    1240.
                  1591.    1666.    1743.    1821.    1901.    1983.    2629.

    ELEVATION=    1717.    1723.    1728.    1733.    1738.    1743.    1748.    1753.    1758.    1763.
                  1768.    1769.    1770.    1771.    1772.    1773.    1780.

                         CREL    SPWID     COQW     EXPW    ELEVL     COQL    CAREA     EXPL
                       1768.0    150.0      3.1      1.5      0.0      0.0      0.0      0.0

                                                 DAM DATA
                                         TOPEL     COQD     EXPD   DAMWID
                                        1771.5      0.0      0.0       0.

     CREST LENGTH        0.      60.     200.     370.     455.     575.     615.     700.
     AT OR BELOW
     ELEVATION       1771.5   1771.6   1771.8   1771.9   1772.1   1772.3   1773.0   1776.8

PEAK OUTFLOW IS     4204. AT TIME  19.08 HOURS
```

```
                                        END-OF-PERIOD FLOW
  MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP Q      MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP Q
                                                                          SUM  18.48  15.86   2.62   306270.
                                                                             ( 469.)( 403.)(  67.)( 8672.59)

        **********      **********      **********      **********      **********

                                     SUB-AREA RUNOFF COMPUTATION

           SUBAREA RESERVOIR RUNOFF
                            ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                           SA-RES       0       0       0       0       0       1       0       0

                                              HYDROGRAPH DATA
               IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
                   1      -1    0.12    0.00   10.00    0.00   0.000       0       1       0

                                              PRECIP DATA
                         SPFE     PMS      R6     R12     R24     R48     R72     R96
                         0.00   17.50  111.00  123.00  132.00    0.00    0.00    0.00
  TRSPC COMPUTED BY THE PROGRAM IS 0.800

                                              LOSS DATA
             LROPT   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
                 0    0.00    0.00    1.00    0.00    0.00    1.00    0.00    0.00    0.00    0.00

                                              RECESSION DATA
                               STRTQ=   -2.00    QRCSN=    0.00    RTIOR= 1.00

                     0                  END-OF-PERIOD FLOW
  MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP Q      MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP Q
                                                                          SUM  18.48  18.48   0.00    16715.
                                                                             ( 469.)( 469.)(   0.)(  473.32)

        **********      **********      **********      **********      **********

                                          COMBINE HYDROGRAPHS

           COMBINING HYDROGRAPHS 1,2
                            ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                            SA-2C       2       0       0       0       0       1       0       0

        **********      **********      **********      **********      **********

                                          HYDROGRAPH ROUTING

           ROUT FLOWS THROUGH RESERVOIR
                            ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                              RES       1       0       0       2       0       1       0       0
                                              ROUTING DATA
                    QLOSS   CLOSS     AVG    IRES   ISAME    IOPT    IPMP            LSTR
                      0.0   0.000    0.00       1       1       0       0               0

                            NSTPS   NSTDL     LAG   AMSKK       X     TSK   STORA  ISPRAT
                                1       0       0   0.000   0.000   0.000  -1768.       0

  SURFACE AREA=        0.      2.      7.     15.     21.     30.     38.     45.     58.     67.
                      74.
```

```
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
 LAST MODIFICATION  26 FEB 79
********************************

RUN  DATE: 6/30/80
     TIME: 7:39 AM

                    NED DAM INSPECTION  CONTRACT: DACW33-80-C-0012    SHOT1
                    VT. 00176, SUGAR HILL DAM, 21-06-79106
                    OVERTOPPING - TEST FLOOD = FULL PMF & HALF PMF

                                        JOB SPECIFICATION
                       NQ       NHR      NMIN      IDAY       IHR      IMIN     METRC      IPLT      IPRT     NSTAN
                      288         0         5         0         0         0         0         0         4         0
                                                  JOPER       NWT     LROPT     TRACE
                                                      5         0         0         0

                                    MULTI-PLAN ANALYSES TO BE PERFORMED
                                          NPLAN= 1 NRTIO= 2 LRTIO= 1

                        RTIOS=    1.00    0.50

              *********          *********          *********          *********          *********

                                        SUB-AREA RUNOFF COMPUTATION

                   SUBAREA 1 RUNOFF COMPUTATION
                                  ISTAQ     ICOMP     IECON     ITAPE      JPLT      JPRT     INAME    ISTAGE     IAUTO
                                   SA-1         0         0         0         0         0         1         0         0

                                               HYDROGRAPH DATA
                     IHYDG      IUHG     TAREA      SNAP     TRSDA     TRSPC     RATIO     ISNOW     ISAME     LOCAL
                         1         1      2.85      0.00     10.00      0.00     0.000         0         1         0

                                                  PRECIP DATA
                                SPFE       PMS        R6       R12       R24       R48       R72       R96
                                0.00     17.50    111.00    123.00    132.00      0.00      0.00      0.00
TRSPC COMPUTED BY THE PROGRAM IS 0.800

                                                   LOSS DATA
                 LROPT     STRKR     DLTKR     RTIOL     ERAIN     STRKS     RTIOK     STRTL     CNSTL     ALSMX     RTIMP
                     0      0.00      0.00      1.00      0.00      0.00      1.00      1.00      0.10      0.00      0.00

                                              UNIT HYDROGRAPH DATA
                                          TP=  3.20      CP=0.63      NTA=  0

                                                 RECESSION DATA
                                   STRTQ=    -2.00      QRCSN=     0.00      RTIOR= 1.00

                  UNIT HYDROGRAPH100 END-OF-PERIOD ORDINATES, LAG=    3.20 HOURS, CP= 0.63  VOL= 0.89
                     2.        6.       13.       21.       30.       40.       51.       63.       75.       87.
                   100.      114.      127.      142.      156.      171.      185.      200.      216.      231.
                   246.      261.      276.      289.      301.      313.      323.      333.      342.      349.
                   356.      362.      367.      371.      374.      376.      377.      377.      375.      372.
                   368.      360.      351.      341.      331.      322.      313.      304.      296.      288.
                   280.      272.      264.      257.      250.      243.      236.      230.      223.      217.
                   211.      205.      199.      194.      188.      183.      178.      173.      168.      164.
                   159.      155.      150.      146.      142.      138.      134.      131.      127.      123.
                   120.      117.      113.      110.      107.      104.      101.       98.       96.       93.
                    90.       88.       86.       83.       81.       79.       76.       74.       72.       70.
```

```
A NED DAM INSPECTION  CONTRACT: DACW33-80-C-0012    SHOT1
A VT. 00176, SUGAR HILL DAM, 21-06-79106
A OVERTOPPING - TEST FLOOD - FULL PMF & HALF PMF
B      288       0       5                                       4
B1       5
J        1       2       1
J1     1.0     0.5
K        0    SA-1                                       1
K1 SUBAREA 1 RUNOFF COMPUTATION
M        1       1   2.855              10                       1
P             17.5     111     123     132
T                                                      1.0     .10
W      3.2    .525
X      -2.       0       1
K        0  SA-RES                                       1
K1 SUBAREA RESERVOIR RUNOFF
M        1      -1    .116              10                       1
P             17.5     111     123     132
T                                                        0       0
U        1
U1     898
X      -2.       0       1
K        2   SA-2C                                       1
K1 COMBINING HYDROGRAPHS 1,2
K        1     RES                       2               1
K1 ROUT FLOWS THROUGH RESERVOIR
Y                                1       1
Y1       1                                           -1768
$A       0     2.0     7.3    14.8    21.2    30.2    37.6    45.4    57.5    66.5
$A    74.1    75.8    77.4    79.1    80.7    82.4   102.7
$E    1717    1723    1728    1733    1738    1743    1748    1753    1758    1763
$E    1768    1769    1770    1771    1772    1773    1780
$$    1768     150   3.087     1.5
$D1771.5
$L       0      60     200     370     455     575     615     700
$V1771.5  1771.6  1771.8  1771.9  1772.1  1772.3  1773.0  1776.8
K       99
A
A
A
A
A
```

G. E. Ainsworth Associates
20 Sugarloaf Street
S. DEERFIELD, MA 01373
Phone 665-2161

JOB SUGAR HILL DAM
SHEET NO. OF
CALCULATED BY ELV DATE 1/25/80
CHECKED BY [illegible] DATE 2/80
SCALE 21-06-79106

DRAINAGE AREA DATA FOR HEC-1DB PROGRAM

SUBAREA 1 : AREA TRIBUTARY DIRECTLY TO SUGAR HILL RESERVOIR
AREA = 2.855 SQUARE MILES

LOSS RATES: 1.0" - INITIALLY
0.1"/HOUR - CONSTANT LOSS RATE

UNIT HYDROGRAPH PARAMETERS : USE SNYDER METHOD

A = DRAINAGE AREA = 2.855 SQUARE MILES
L = LENGTH OF MAIN WATERCOURSE TO UPSTREAM LIMIT OF DRAINAGE AREA = 2.75 MILES
L_{CA} = LENGTH OF MAIN WATERCOURSE TO POINT OPPOSITE THE CENTROID OF THE DRAINAGE AREA = 1.75 MILES
C_t = SNYDER'S BASIN COEFFICIENT = 2.0 ASSUMED AVERAGE
C_p = SNYDER'S PEAKING COEFFICIENT = 0.625 ASSUMED AVERAGE
t_p = STANDARD LAG IN HOURS = $C_t(LL_{CA})^{0.3}$ = 3.2 HOURS

∴ USE t_p = 3.2 HOURS

SUBAREA 2 : SUGAR HILL RESERVOIR SURFACE, AREA = 0.116 SQUARE MILES (74.1 ACRES)

LOSS RATES : NONE BECAUSE RAINFALL = RUNOFF FOR WATER SURFACE

UNIT HYDROGRAPH PARAMETERS :

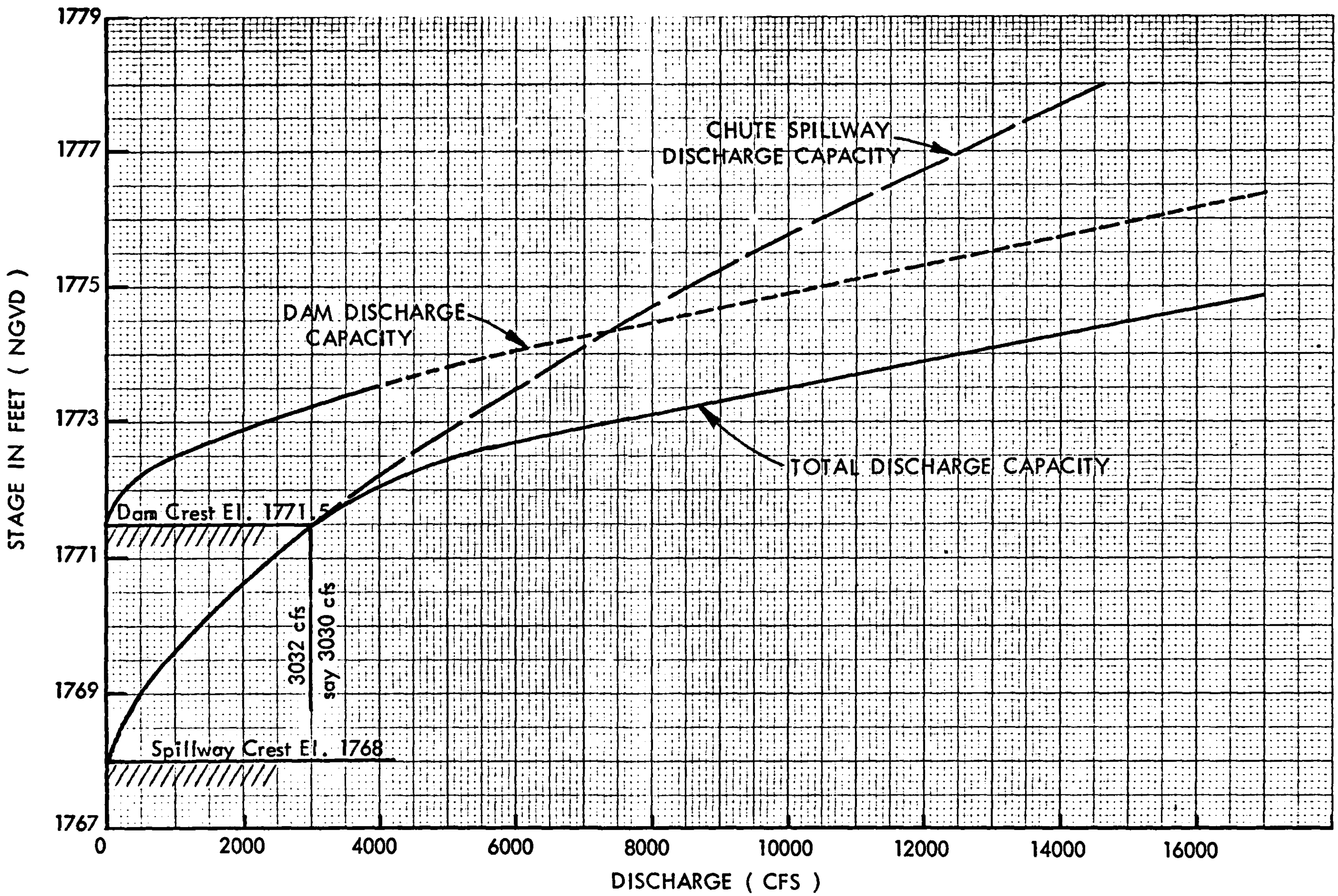
FOR U.H. W/ 5 MINUTE DURATION + 1" RAIN

$$\bar{Q} = \frac{A(1")}{t} = \frac{.116\ mi^2\ (1")}{5\ minutes}\left(\frac{43560\ sq.\ ft.}{1\ acre}\right)\left(\frac{1'}{12"}\right)\left(\frac{1\ minute}{60\ seconds}\right)\left(\frac{640\ acres}{1\ mi^2}\right)$$

$\bar{Q}$ = 898 cfs (SINCE NO LOSS RATE)

SUGAR HILL DAM, GOSHEN, VERMONT

STAGE — DISCHARGE



G. E. Ainsworth Associates
20 Sugarloaf Street
S. DEERFIELD, MA 01373
Phone 665-2161

JOB SUGAR HILL DAM
SHEET NO. ____ OF ____
CALCULATED BY ELV DATE 1/25/80
CHECKED BY [illegible] DATE 2/80
SCALE 21-06-79106

DISCHARGE COMPUTATIONS

DAM APPURTENANCE	ELEVATION (NGVD)	SIZE
SPILLWAY	CREST EL. = 1768	150' TOTAL LENGTH W/ A 100' + A 50' WEIR CREST
DAM	CREST EL. = 1771.5 (Low Point, Non-Level)	700' CREST LENGTH W/O SPILLWAY
OUTLET CONDUIT	INVERT IN EL. = 1707	4'x4' BOX CULVERT W/ VARIOUS CONTROLS (SEE TEXT)

FOR FLOW OVER SPILLWAY ONLY: $Q = 3.087 LH^{1.5}$

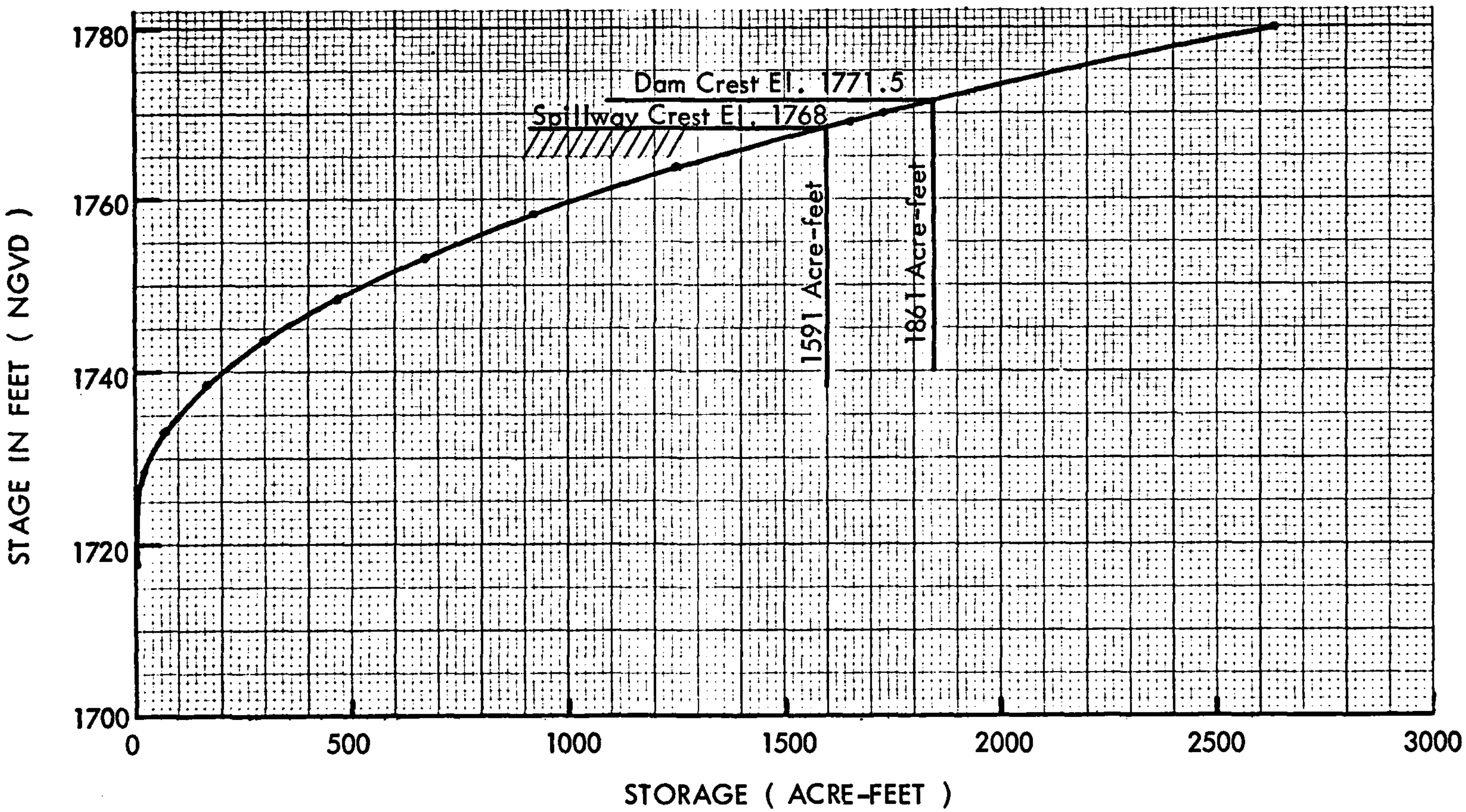
(FORMULA FOR CRITICAL FLOW OVER A BROAD-CRESTED WEIR. INPUTED INTO HEC-1DB COMPUTER PROGRAM FOR DISCHARGE COMPUTATIONS. REFERENCE 9)

	ELEVATION (NGVD)	$H_{SPILLWAY}$ (feet)	H_{DAM} (feet)	$Q_{SPILLWAY}$ (cfs)	Q_{DAM}* (cfs)	$Q_{OUTLET WORKS}$ (cfs)	Q_T (cfs)
Spillway Crest →	1768	0	0	0	0	0	0
	1769	1	0	463	0	ALL OUTLETS ASSUMED CLOSED	463
	1770	2	0	1310	0		1310
	1771	3	0	2406	0		2406
Dam Crest →	1771.5	3.5	0.5	3032	0		3032 SAY 3030
	1772	4	1.5	3704	159		3863
	1773	5	2.5	5177	2262		7439
	1774	6	3.5	6805	5761		12566
	1775	7	4.5	8576	10446	0	19022

* CALCULATED W/ HEC-1DB PROGRAM FOR FLOW OVER NON-LEVEL DAM CREST.

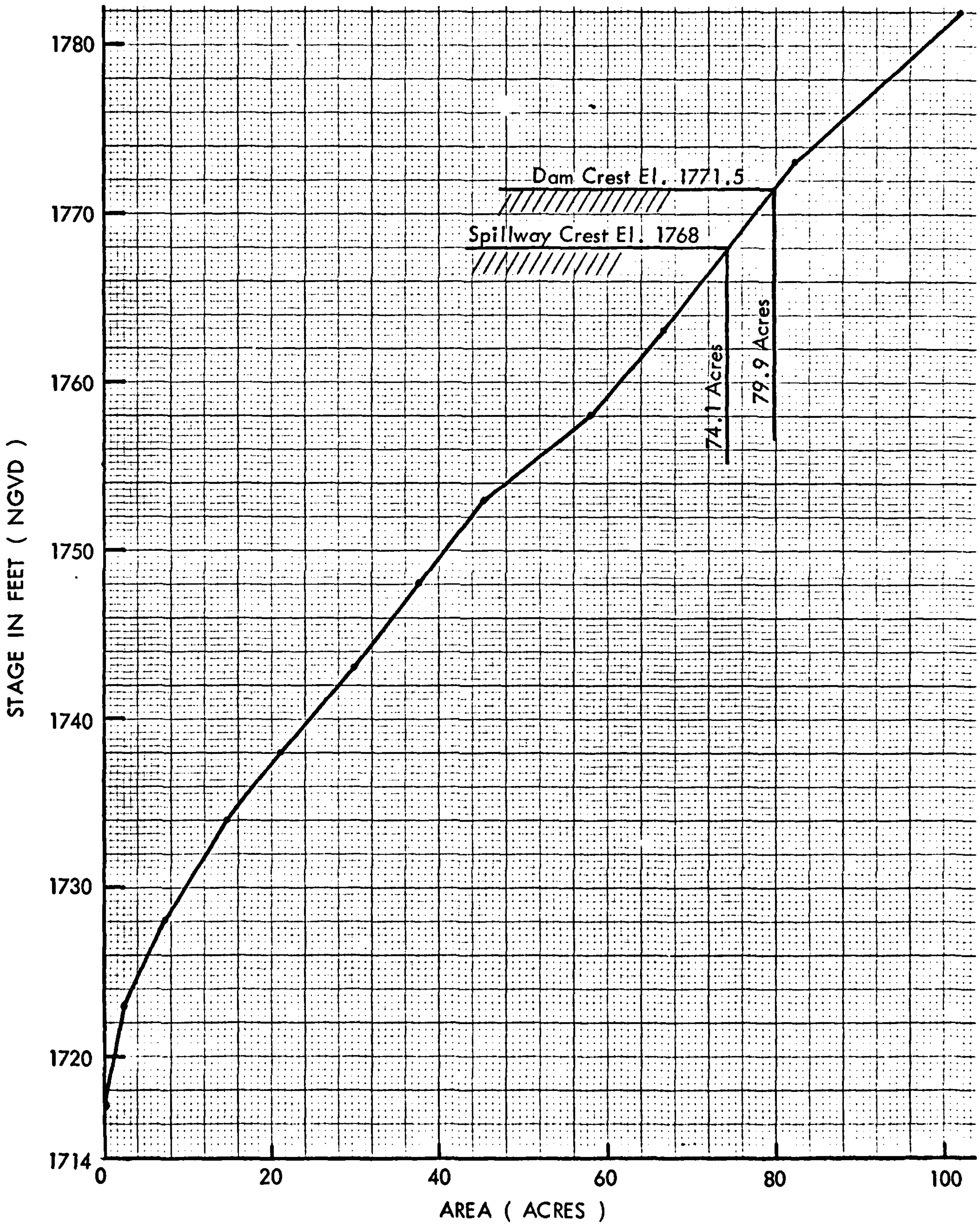
SUGAR HILL DAM, GOSHEN, VERMONT

STAGE — STORAGE



SUGAR HILL DAM, GOSHEN, VERMONT

STAGE — AREA



G. E. Ainsworth Associates
20 Sugarloaf Street
S. DEERFIELD, MA 01373
Phone 665-2161

JOB SUGAR HILL DAM
SHEET NO. OF
CALCULATED BY ELV DATE 1/25/80
CHECKED BY [illegible] DATE 2/80
SCALE 21-06-79106

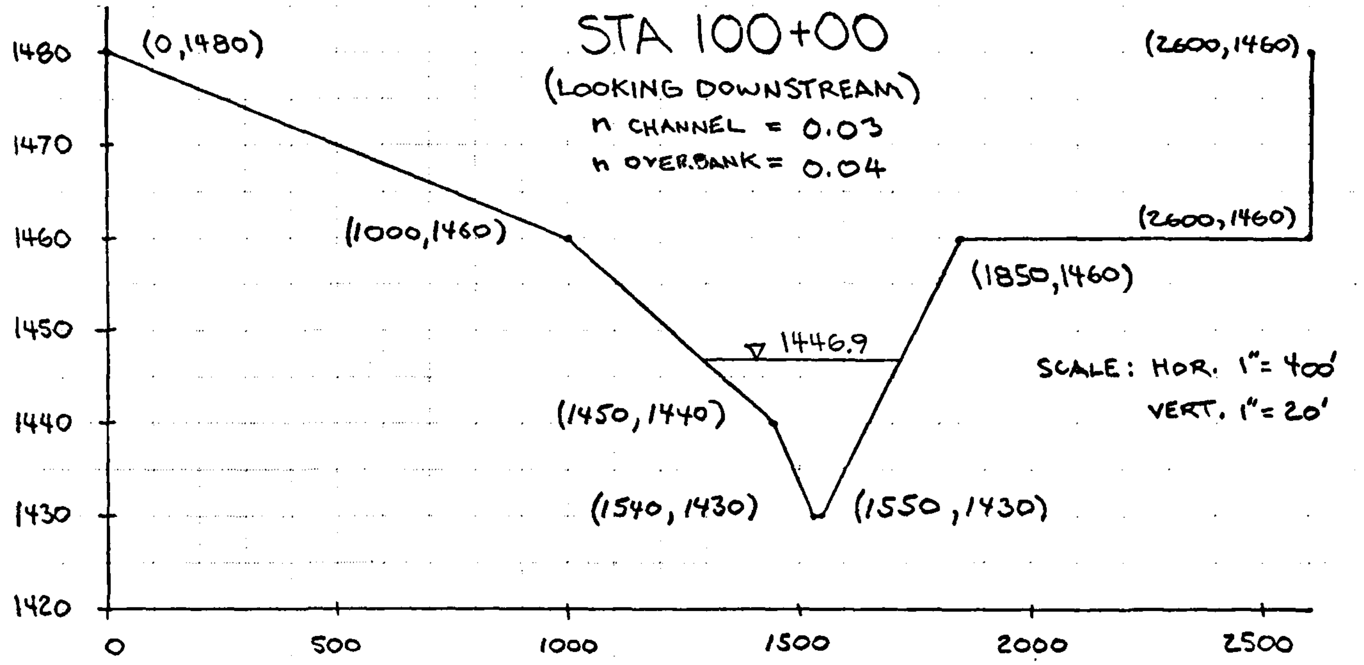
ELEVATION - AREA - STORAGE COMPUTATIONS

DRAINAGE AREA

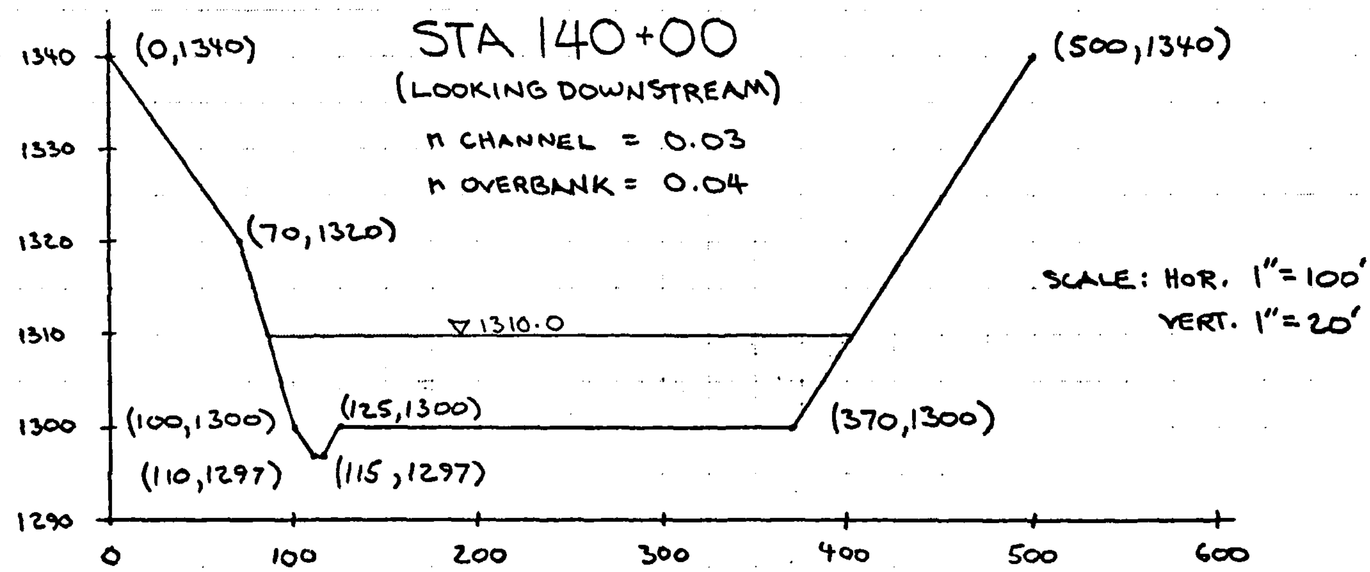
	AREA (acres)	AREA (Square miles)
RESERVOIR SURFACE (SUB AREA 2) @ NORMAL POOL EL = 1768	74.1	0.116
WATERSHED DIRECT TO RESERVOIR (SUB AREA 1)	1827.2	2.855
DRAINAGE AREA TO SUGAR HILL DAM	1901.3	2.971

GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB SUGAR HILL DAM
SHEET NO. ______ OF ______
CALCULATED BY ELV DATE 2/1/80
CHECKED BY FAB DATE 2/80
SCALE 21-06-79106



FLOW AREA $= \frac{S}{L} = \frac{369}{5000} \times 43560 = 3215$ SF



FLOW AREA $= \frac{S}{L} = \frac{273}{4000} \times 43560 = 2973$ SF

GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

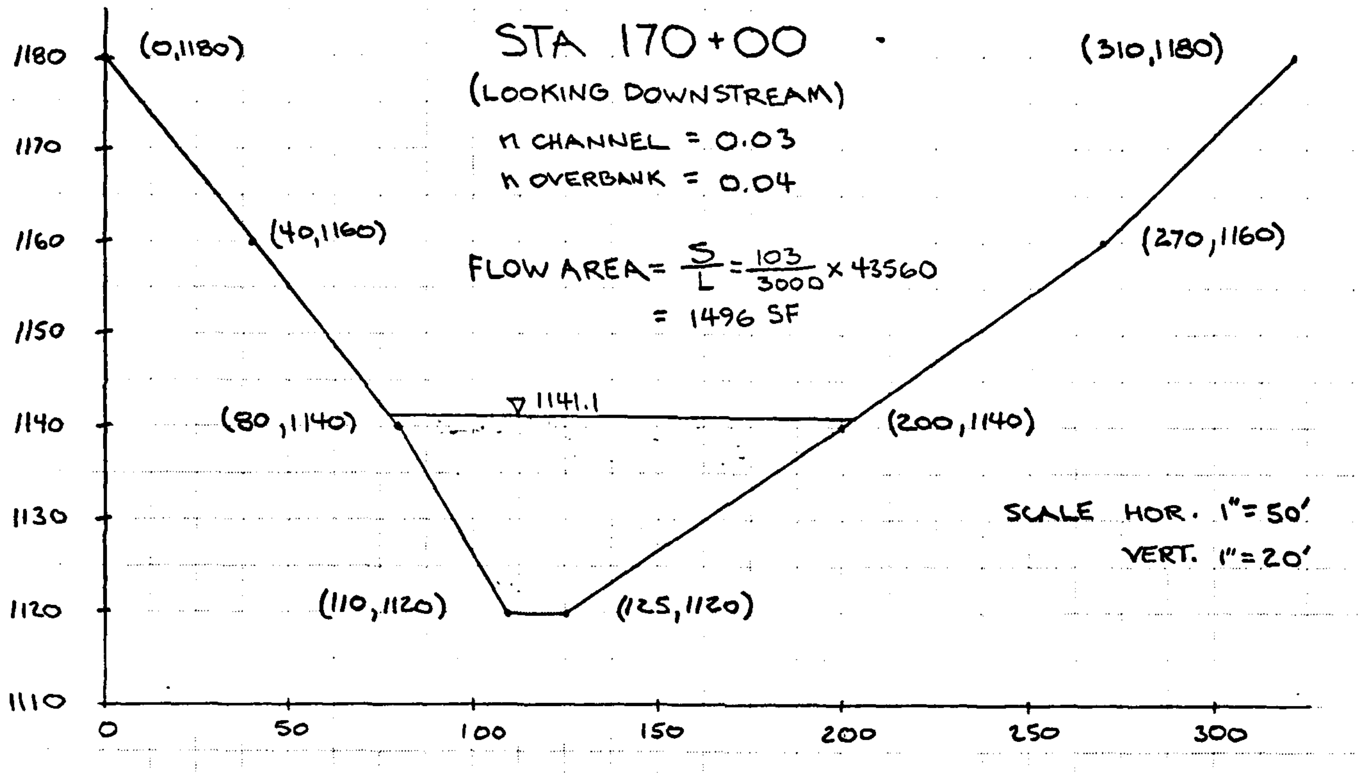
JOB SUGAR HILL DAM

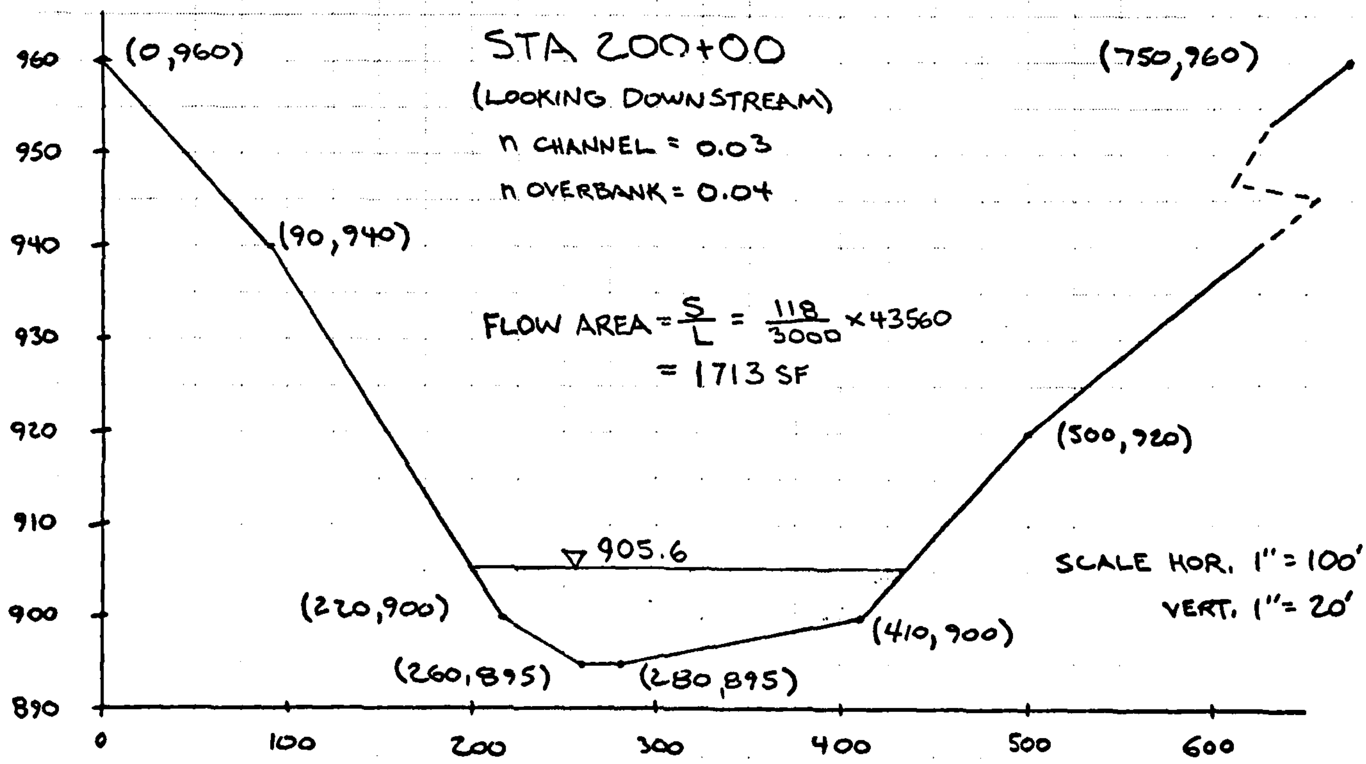
SHEET NO. ______ OF ______

CALCULATED BY ELV DATE 2/1/80

CHECKED BY [signature] DATE 2/80

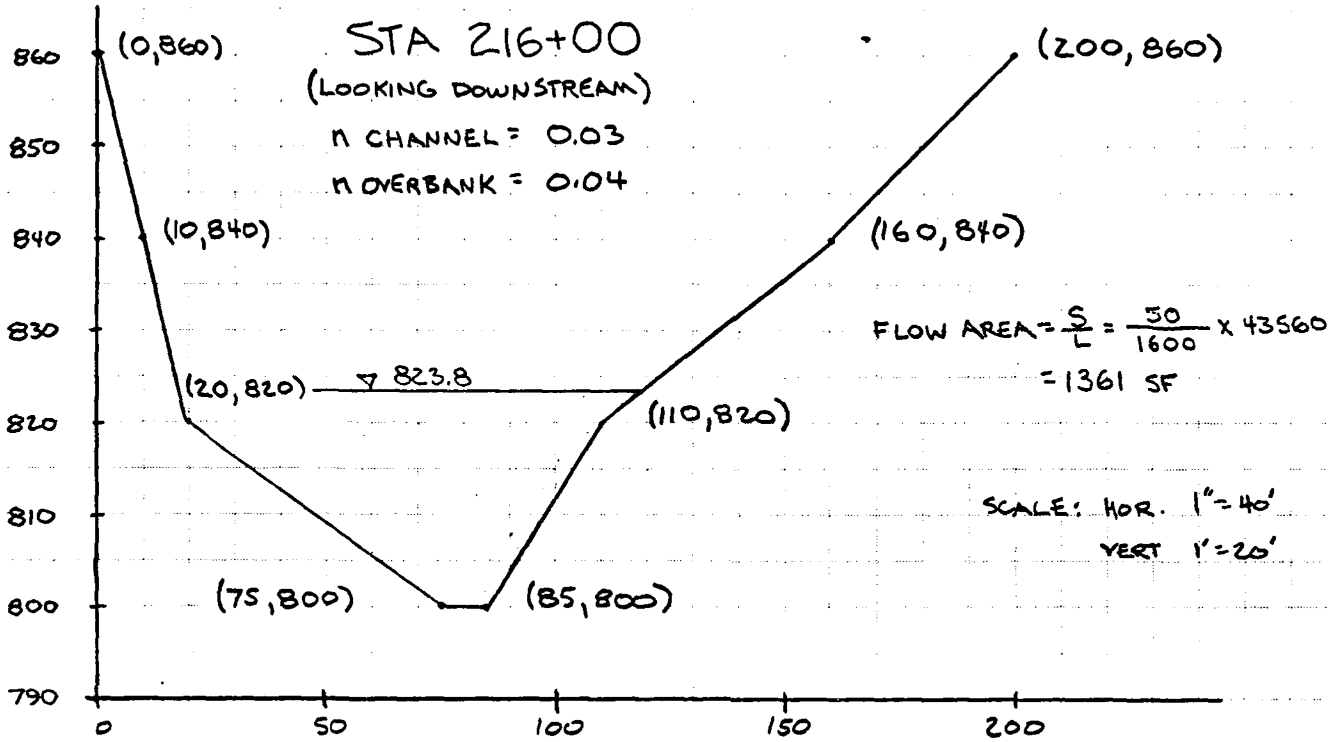
SCALE 21-06-79106

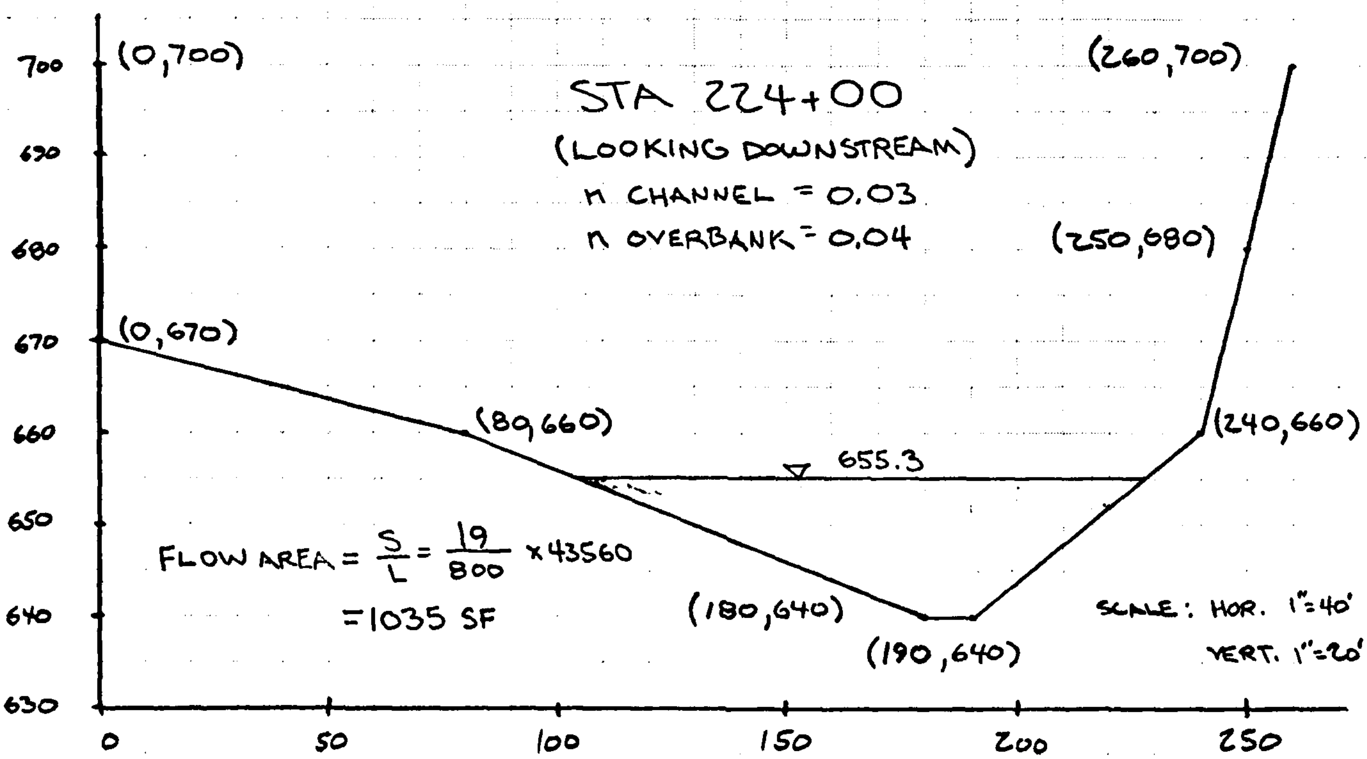




GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

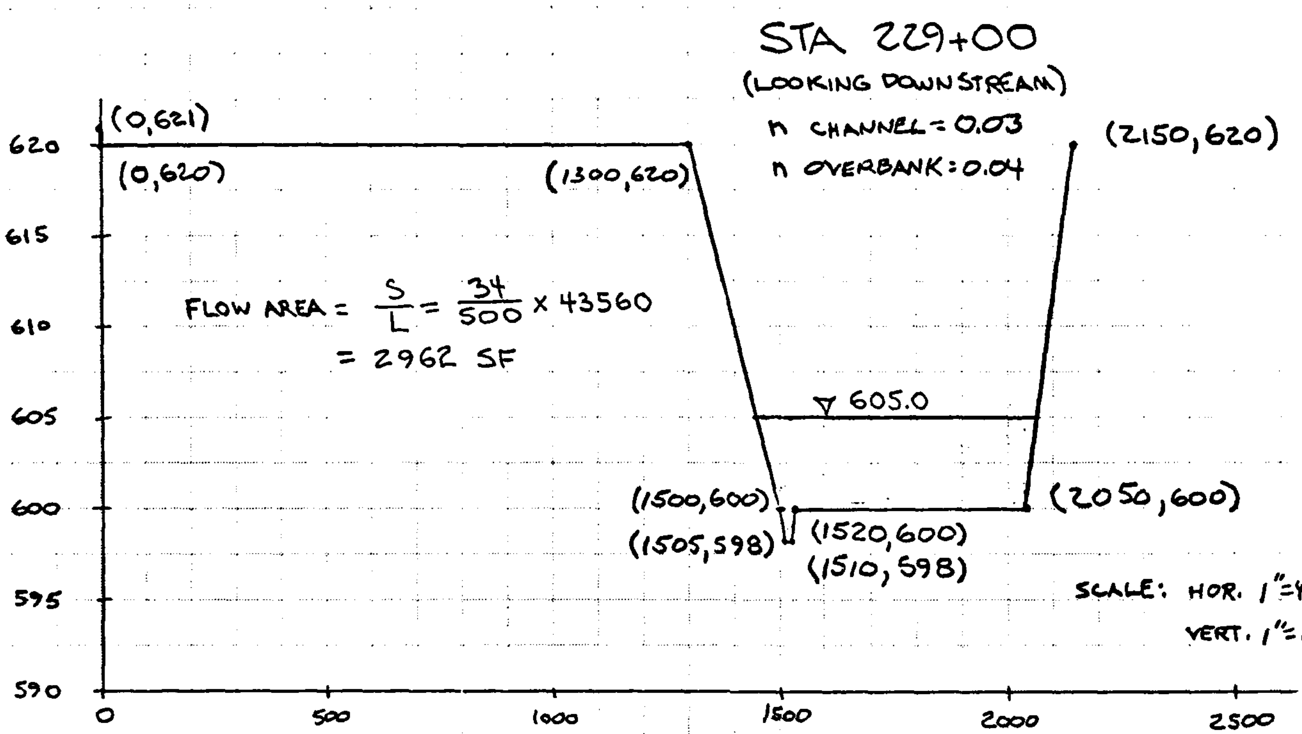
JOB SUGAR HILL DAM
SHEET NO. ______ OF ______
CALCULATED BY ELV DATE 2/1/80
CHECKED BY [illegible] DATE 2/80
SCALE 21-06-79106



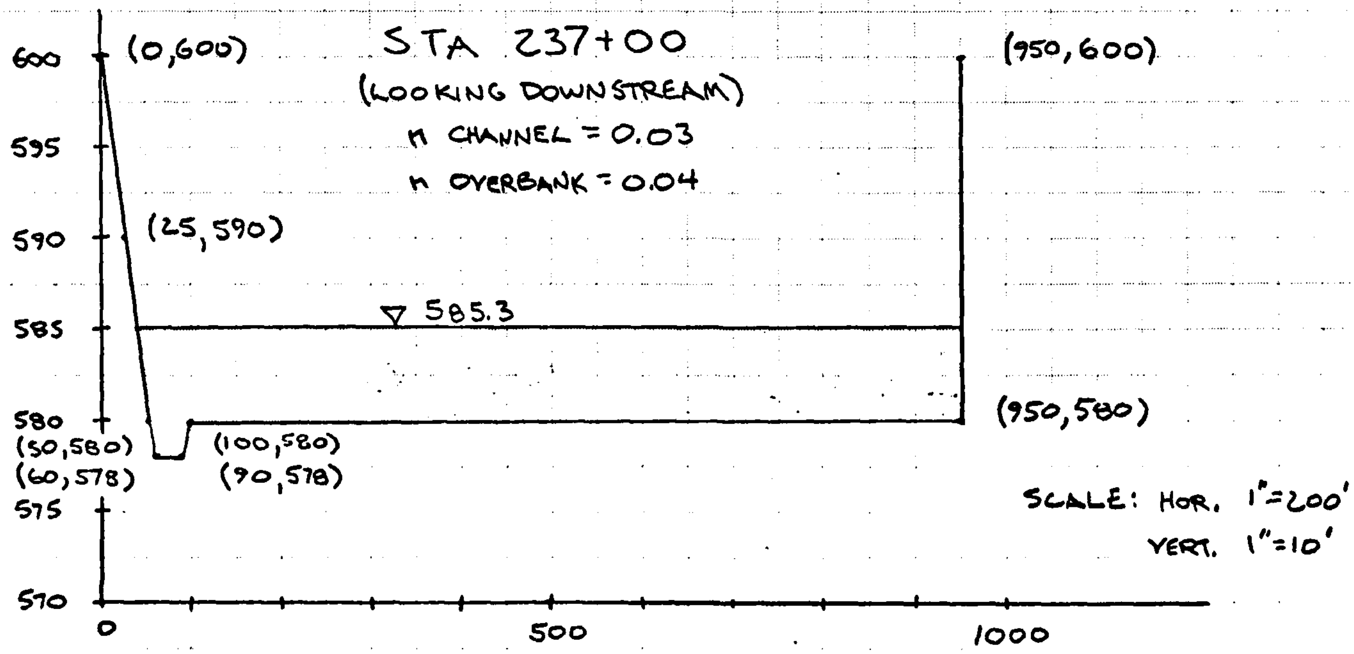


GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB: SUGAR HILL DAM
SHEET NO. ______ OF ______
CALCULATED BY: ELV DATE: 2/1/80
CHECKED BY: [illegible] DATE: 2/80
SCALE: 21-06-79106



FLOW AREA = $\frac{S}{L} = \frac{34}{500} \times 43560$
$= 2962$ SF



FLOW AREA = $\frac{S}{L} = \frac{90}{800} \times 43560 = 4901$ SF

G. E. Ainsworth Associates
20 Sugarloaf Street
S. DEERFIELD, MA 01373
Phone 665-2161

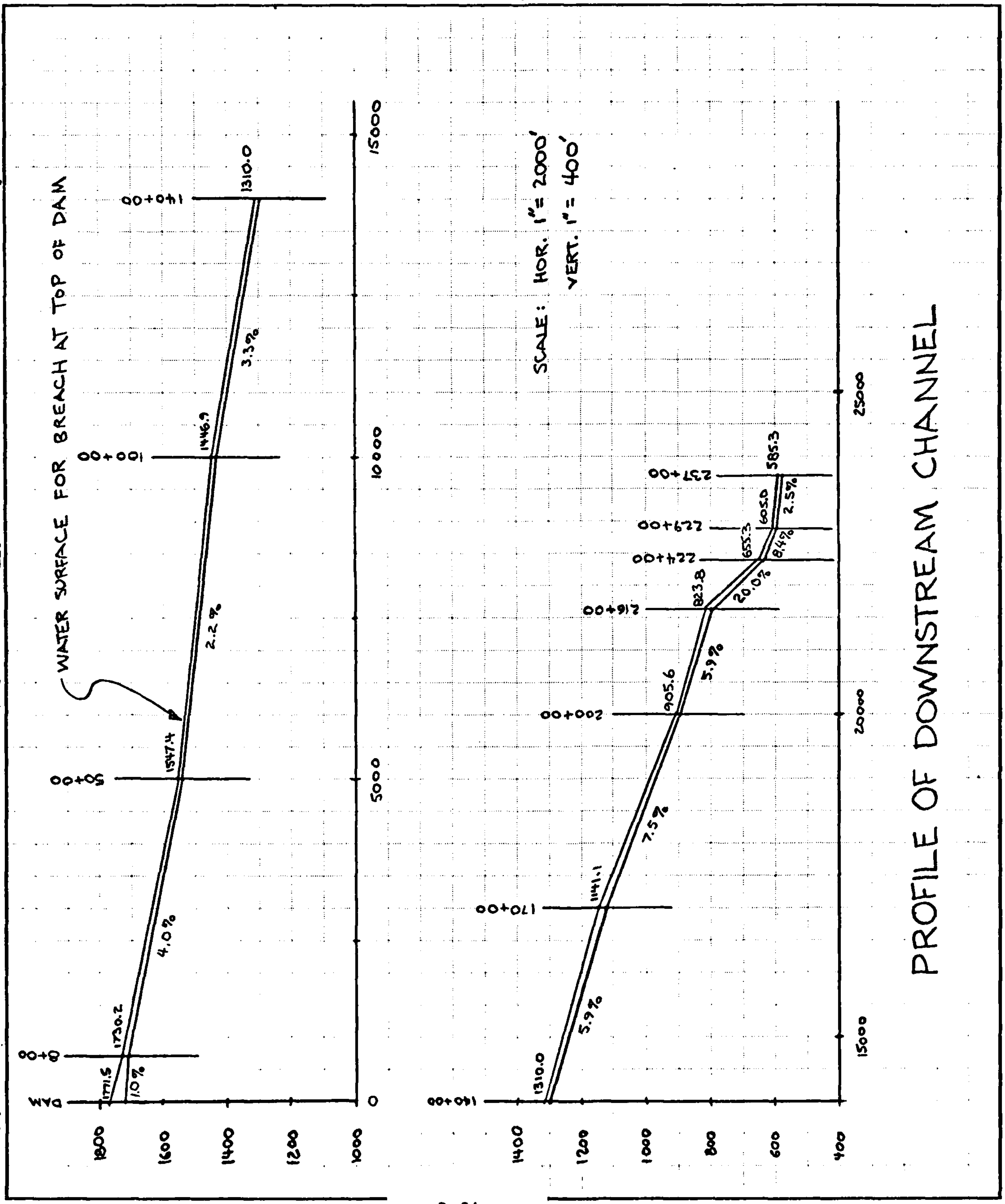
JOB SUGAR HILL DAM

SHEET NO. ______ OF ______

CALCULATED BY ELV DATE 2/1/80

CHECKED BY JPB DATE 2/80

SCALE 21-06-79106



```
A NED DAM INSPECTION  CONTRACT: DACW33-80-C-0012    SHPF1
A VT. 00176. SUGAR HILL DAM. 21-06-79106
A PRIOR FLOW AT TOP OF DAM
B     180       0       1                                               4
B1      5
J       1       1       1
J1    1.0
K       0  SA-RES                                       1
K1 SUBAREA RESERVOIR RUNOFF
M       1      -1    .116                                               1
P
T
J       1
U1
X    3032       0       1
K       1     RES                                       1
K1 ROUT FLOWS THROUGH RESERVOIR
Y                               1       1
Y1      1                                         -1771.5
$A      0     2.0     7.3    14.8    21.2    30.2    37.6    45.4    57.5    66.5
$A   74.1    75.8    77.4    79.1    80.7    82.4   102.7
$E   1717    1723    1728    1733    1738    1743    1748    1753    1758    1763
$E   1768    1769    1770    1771    1772    1773    1780
$$   1769     150   3.087     1.5
$D1771.5
$L      0      60     200     370     455     575     615     700
$V1771.5  1771.6  1771.8  1771.9  1772.1  1772.3  1773.0  1776.8
K       1    7+00                                3      1
K1 CHANNEL ROUTING STA 7+00
Y                               1       1
Y1      1
Y6    .04     .03     .04    1710    1740     700     .01
Y7      0    1750      50    1740     150    1720     240    1710     250    1710
Y7    310    1720     560    1720     650    1740
K       1   50+00                                3      1
K1 CHANNEL ROUTING STA 50+00
Y                               1       1
Y1      1
Y6    .04     .03     .04    1540    1580    4300    .040
Y7      0    1580      60    1560     140    1544     160    1540     170    1540
Y7    200    1544    1850    1544    1850    1580
K       1  100+00                                3      1
K1 CHANNEL ROUTING STA 100+00
Y                               1       1
Y1      1
Y6    .04     .03     .04    1430    1480    5000    .022
Y7      0    1480    1000    1460    1450    1440    1540    1430    1550    1430
Y7   1850    1460    2600    1460    2600    1480
K       1  140+00                                3      1
K1 CHANNEL ROUTING STA  140+00
Y                               1       1
Y1      1
Y6    .04     .03     .04    1297    1340    4000    .033
Y7      0    1340      70    1320     100    1300     110    1297     115    1297
Y7    125    1300     370    1300     500    1340
K       1  170+00                                3      1
K1 CHANNEL ROUTING STA  170+00
Y                               1       1
Y1      1
Y6    .04     .03     .04    1120    1180    3000    .059
```

```
Y7       0   1180     40   1160     80   1140    110   1120    125   1120
Y7     29[illegible]  11[illegible]    270   1160    310   1180
K        [illegible]  20[illegible]                          5      1
K1 CHAN[illegible] [illegible]ING STA 200+00
Y                                   1      1
Y1       [illegible]
Y6     .0[illegible]    [illegible]    .04    [illegible]    960   3000   .075
Y7       [illegible]       [illegible]    [illegible]    220    900    260    895    280    895
Y7     [illegible]                                   460
K                                                   [illegible]      1
K1 CHAN[illegible] STA [illegible]
Y                                          1
Y1       [illegible]
Y5     .0[illegible]                                [illegible]   [illegible]   .0[illegible]9
Y7       [illegible]                                       [illegible]     75    800     85    800
Y7     11[illegible]                                       [illegible]
K        1  22[illegible]                                   [illegible]      1
K1 CHAN[illegible]
Y
Y1       1
Y5     .04    .0[illegible]                                       .20
Y7       9    70[illegible]                                        [illegible]    [illegible]    19[illegible]    640
Y7     240    660
K        1  229+00
K1 CHANNEL ROUTING STA [illegible]
Y
Y1       [illegible]
Y[illegible]    .[illegible]    .03    .04
              [illegible]21   1300                              [illegible]    [illegible]    [illegible]    [illegible]
              [illegible]   2050
           [illegible]
K1 [illegible]ING STA 237+00
                                     1
                        .04    578
                         25    590                              [illegible]    [illegible]    [illegible]
                         [illegible]    580
```

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS RATIO 1.00
HYDROGRAPH AT	SA-RES	0.12 (0.30)	1	3032. (85.86)(
ROUTED TO	RES	0.12 (0.30)	1	3032. (85.86)(
ROUTED TO	7+00	0.12 (0.30)	1	3032. (85.86)(
ROUTED TO	50+00	0.12 (0.30)	1	3032. (85.86)(
ROUTED TO	100+00	0.12 (0.30)	1	3032. (85.86)(
ROUTED TO	140+00	0.12 (0.30)	1	3032. (85.86)(
ROUTED TO	170+00	0.12 (0.30)	1	3032. (85.86)(
ROUTED TO	200+00	0.12 (0.30)	1	3032. (85.86)(
ROUTED TO	216+00	0.12 (0.30)	1	3032. (85.86)(
ROUTED TO	224+00	0.12 (0.30)	1	3032. (85.86)(
ROUTED TO	229+00	0.12 (0.30)	1	3032. (85.86)(
ROUTED TO	237+00	0.12 (0.30)	1	3032. (85.86)(

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1771.50	1768.00	1771.50
STORAGE	1861.	1591.	1861.
OUTFLOW	3032.	0.	3032.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	1771.50	0.00	1861.	3032.	0.00	0.00	0.00

PLAN 1 STATION 7+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	3032.	1715.5	0.20

PLAN 1 STATION 50+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	3032.	1543.7	2.43

PLAN 1 STATION 100+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	3032.	1434.1	2.45

PLAN 1 STATION 140+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	3032.	1300.6	2.58

PLAN 1 STATION 170+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	3032.	1124.2	2.62

PLAN 1 STATION 200+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	3032.	896.8	2.65

PLAN 1 STATION 216+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	3032.	805.2	2.67

PLAN 1 STATION 224+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	3032.	643.4	2.67

PLAN 1 STATION 229+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	3032.	600.5	2.68

PLAN 1 STATION 237+00

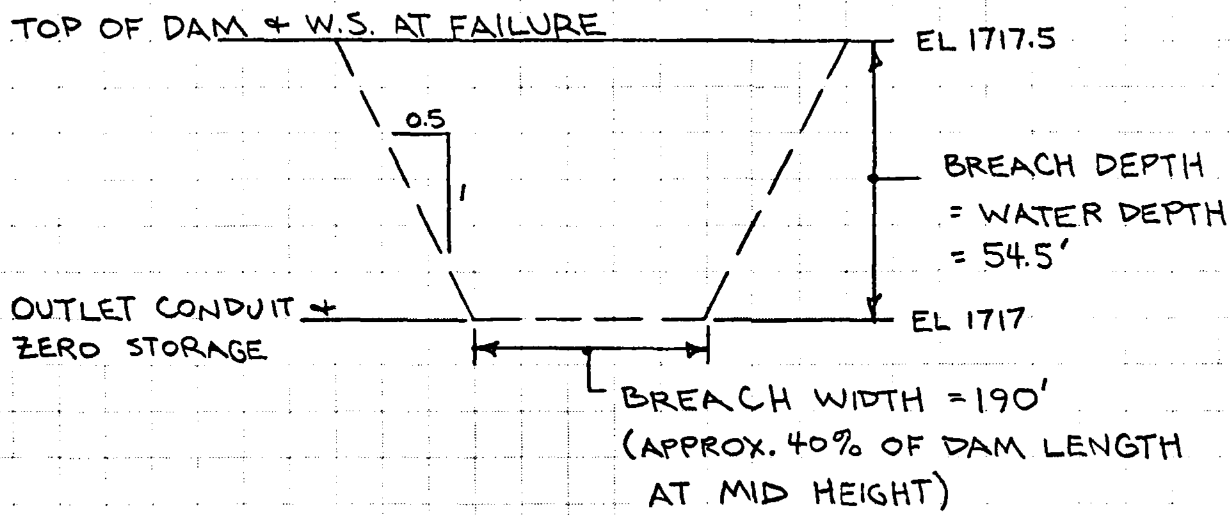
RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	3032.	580.4	2.72

G. E. Ainsworth Associates
20 Sugarloaf Street
S. DEERFIELD, MA 01373
Phone 665-2161

JOB SUGAR HILL DAM
SHEET NO. ____ OF ____
CALCULATED BY ELV DATE 7/2/80
CHECKED BY GPB DATE 7/80
SCALE 21-06-79106

BREACH CRITERIA

EARTH DAM, NO CORE WALL



RULE OF THUMB PEAK OUTFLOW

$$Q_p = \frac{8}{27} W_b \sqrt{g}\, Y_o^{1.5}$$

W_b = BREACH WIDTH = 190'
Y_o = WATER DEPTH = 54.5'

Q_p = 128,500 cfs ±

HEC-1DB BREACH PROGRAM

CALCULATION INTERVAL = 1 minute

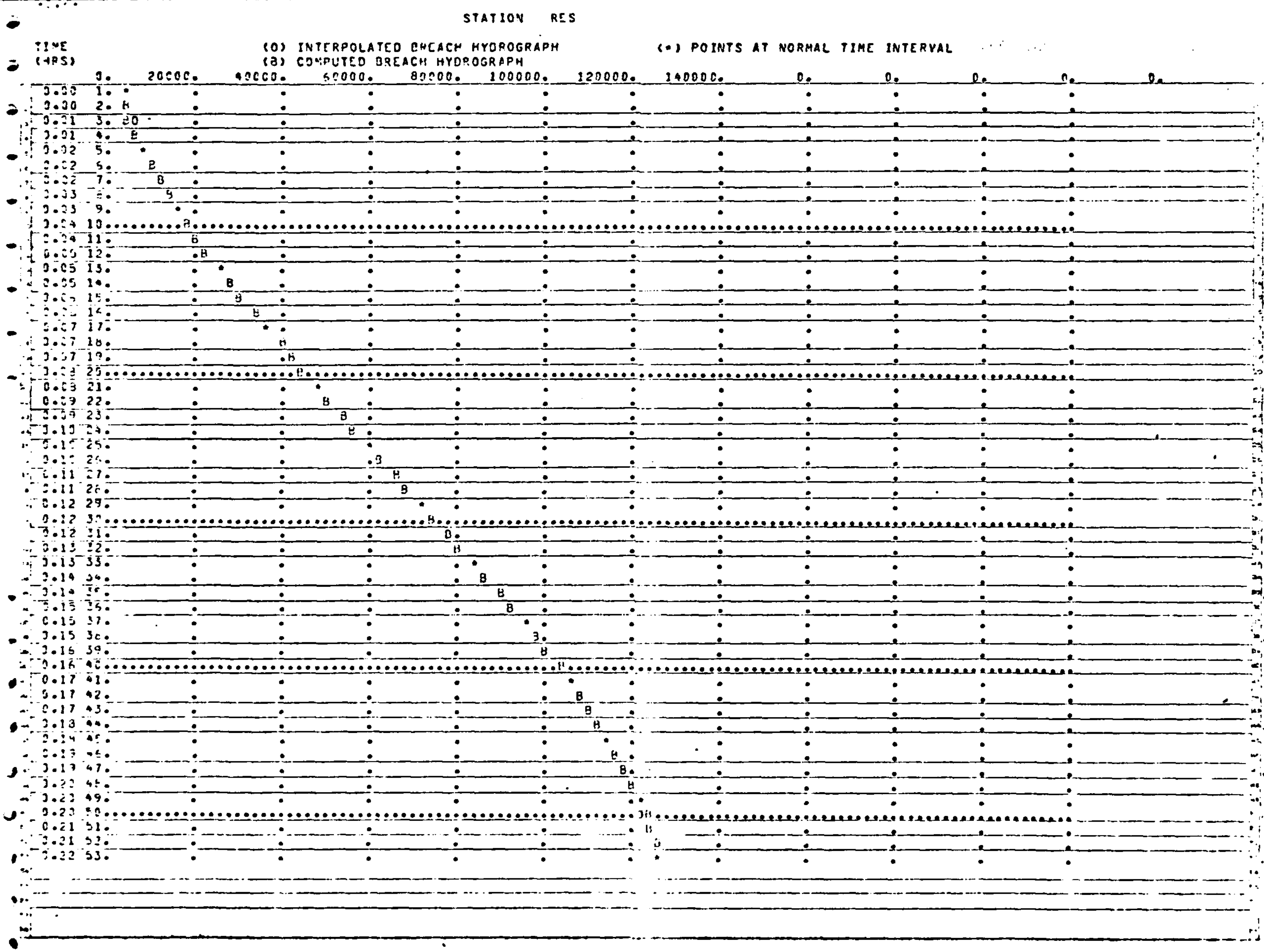
BREACH TIME	PEAK OUTFLOW (cfs)	
0.25 hr.	119,100	
0.20 hr.	140,900	
0.23 hr. = 13.8 min USE	127,100	SAY 127,000
0.22 hr.	131,400	

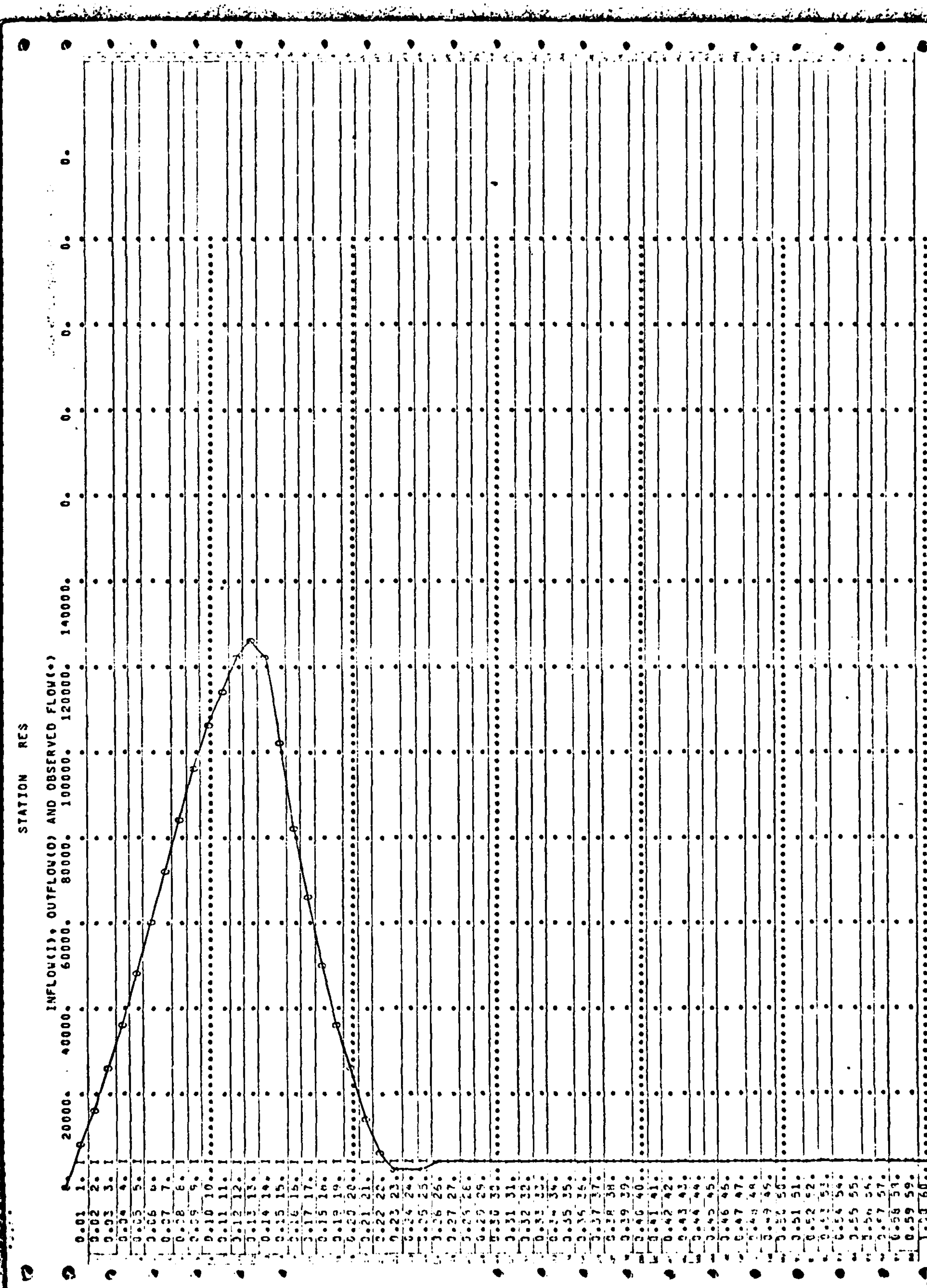
```
A RED DAM INSPECTION  CONTRACT: DACW33-80-C-0012    SHDB1.
A VT. 00176, SUGAR HILL DAM, 21-06-79106
A BREACH AT TOP OF DAM
B     60       0       1                                                4
B1     5
J      1       1       1
J1   1.0
K      0  SA-PES                       1                1
K1 SUBAREA RESERVOIR RUNOFF
M      1      -1    .116                                                1
P
T
O      1
O1
X   3032       0       1
K      1     RES                       2       1        1
K1 ROUT FLOWS THROUGH RESERVOIR
Y                              1       1
Y1     1                                       -1771.5
SA     0     2.0     7.3    14.8    21.2    30.2    37.6    45.4    57.5    66.5
SA  74.1    75.8    77.4    79.1    80.7    82.4   102.7
SE  1717    1723    1728    1733    1738    1743    1748    1753    1758    1763
SE  1768    1769    1770    1771    1772    1773    1780
SS  1769     150   3.087     1.5
SS1771.5
SC     0      60     200     370     455     575     615     700
SV1771.5  1771.6  1771.8  1771.9  1772.1  1772.3  1773.0  1776.8
SD   190      .5    1717     .23  1771.5  1771.5
K      1    7+00                               3        1
K1 CHANNEL ROUTING STA 7+00
Y                              1       1
Y1     1
Y5   .04     .03     .04    1710    1740     700     .01
Y7     0    1763      50    1740     150    1720     240    1710     250    1710
Y7   310    1720     560    1720     650    1740
K      1   50+00                               3        1
K1 CHANNEL ROUTING STA 50+00
Y                              1       1
Y1     1
Y5   .04     .03     .04    1540    1580    4300    .040
Y7     0    1560      60    1560     140    1544     160    1540     170    1540
Y7   200    1544    1850    1544    1850    1580
K      1  100+00                               3        1
K1 CHANNEL ROUTING STA 100+00
Y                              1       1
Y1     1
Y6   .04     .03     .04    1430    1480    5000    .022
Y7     0    1480    1300    1460    1450    1440    1540    1430    1550    1430
Y7  1650    1460    2600    1460    2600    1480
K      1  140+00                               3        1
K1 CHANNEL ROUTING STA  140+00
Y                              1       1
Y1     1
Y5   .04     .03     .04    1297    1340    4000    .033
Y7     0    1340      70    1320     100    1300     110    1297     115    1297
Y7   125    1300     370    1300     500    1340
K      1  170+00                               3        1
K1 CHANNEL ROUTING STA  170+00
Y                              1       1
Y1     1
```

```
Y5    .04     .03     .04    1120    1180    3000    .059
Y7      0    1180      40    1160      80    1140     110    1120     125    1120
Y7    200    1140     270    1160     310    1180
K       1  200+00                                3       1
K1 CHANNEL ROUTING STA 200+00
Y                                 1       1
Y1      1
Y5    .04     .03     .04     895     960    3000    .075
Y7      0     960      90     940     220     900     260     895     280     895
Y7    410     900     500     920     750     960
K       1  216+00                                3       1
K1 CHANNEL ROUTING STA 216+00
Y                                 1       1
Y1      1
Y6    .04     .03     .04     800     860    1600    .059
Y7      0     860      10     840      20     820      75     800      85     800
Y7    110     820     160     840     200     860
K       1  224+00                                3       1
K1 CHANNEL ROUTING STA 224+00
Y                                 1       1
Y1      1
Y5    .04     .03     .04     640     700     800     .20
Y7      0     700       0     670      80     660     180     640     190     640
Y7    240     660     250     680     260     700
K       1  229+00                                3       1
K1 CHANNEL ROUTING STA 229+00
Y                                 1       1
Y1      1
Y5    .04     .03     .04     598     621     500    .084
Y7      0     621    1300     620    1500     600    1505     598    1510     598
Y7   1520     600    2150     600    2150     621
K       1  237+00                                3       1
K1 CHANNEL ROUTING STA 237+00
Y                                 1       1
Y1      1
Y5    .04     .03     .04     578     600     800    .025
Y7      0     600      25     590      50     580      60     578      90     578
Y7    100     590     950     580     950     600
K      99
A
A
A
A
A
```

... HYDROGRAPH WAS DEVELOPED USING A TIME INTERVAL OF 0.004 HOURS DURING BREACH FORMATION.
DOWNSTREAM CALCULATIONS WILL USE A TIME INTERVAL OF 0.017 HOURS.
THIS TABLE COMPARES THE HYDROGRAPH FOR DOWNSTREAM CALCULATIONS WITH THE COMPUTED BREACH HYDROGRAPH.
INTERMEDIATE FLOWS ARE INTERPOLATED FROM END-OF-PERIOD VALUES.

TIME (HOURS)	TIME FROM BEGINNING OF BREACH (HOURS)	INTERPOLATED BREACH HYDROGRAPH (CFS)	- COMPUTED BREACH HYDROGRAPH (CFS)	= ERROR (CFS)	ACCUMULATED ERROR (CFS)	ACCUMULATED ERROR (AC-FT)
0.000	0.000	3032.	3032.	0.	0.	0.
0.004	0.004	4165.	3604.	560.	560.	0.
0.008	0.008	5297.	4647.	651.	1211.	0.
0.012	0.012	6430.	5988.	442.	1654.	1.
0.017	0.017	7563.	7563.	0.	1654.	1.
0.021	0.021	9563.	9333.	229.	1883.	1.
0.025	0.025	11562.	11271.	292.	2175.	1.
0.029	0.029	13562.	13353.	210.	2384.	1.
0.033	0.033	15562.	15562.	0.	2384.	1.
0.037	0.037	18020.	17884.	137.	2521.	1.
0.042	0.042	20479.	20304.	175.	2696.	1.
0.046	0.046	22937.	22811.	126.	2822.	1.
0.050	0.050	25396.	25396.	0.	2822.	1.
0.054	0.054	28129.	28048.	81.	2903.	1.
0.058	0.058	30863.	30760.	103.	3006.	1.
0.062	0.062	33596.	33523.	73.	3079.	1.
0.067	0.067	36330.	36330.	0.	3079.	1.
0.071	0.071	39217.	39174.	43.	3122.	1.
0.075	0.075	42104.	42051.	53.	3175.	1.
0.079	0.079	44991.	44954.	37.	3213.	1.
0.083	0.083	47878.	47876.	0.	3213.	1.
0.087	0.087	50841.	50821.	20.	3232.	1.
0.092	0.092	53803.	53778.	25.	3258.	1.
0.096	0.096	56766.	56748.	18.	3276.	1.
0.100	0.100	59728.	59728.	0.	3276.	1.
0.104	0.104	62760.	62719.	41.	3317.	1.
0.108	0.108	65792.	65725.	67.	3383.	1.
0.112	0.112	68824.	68758.	65.	3449.	1.
0.117	0.117	71856.	71855.	0.	3449.	1.
0.121	0.121	74883.	74937.	-54.	3395.	1.
0.125	0.125	77910.	77985.	-75.	3320.	1.
0.129	0.129	80937.	80995.	-58.	3261.	1.
0.133	0.133	83965.	83965.	0.	3261.	1.
0.137	0.137	86804.	86886.	-82.	3180.	1.
0.142	0.142	89644.	89756.	-112.	3068.	1.
0.146	0.146	92483.	92569.	-86.	2981.	1.
0.150	0.150	95323.	95323.	0.	2981.	1.
0.154	0.154	97934.	98013.	-109.	2872.	1.
0.158	0.158	100465.	100633.	-148.	2724.	1.
0.162	0.162	103066.	103180.	-114.	2610.	1.
0.167	0.167	105647.	105647.	0.	2610.	1.
0.171	0.171	107876.	108024.	-149.	2462.	1.
0.175	0.175	110104.	110305.	-202.	2260.	1.
0.179	0.179	112332.	112487.	-154.	2106.	1.
0.183	0.183	114561.	114561.	0.	2106.	1.
0.187	0.187	116341.	116524.	-183.	1923.	1.
0.192	0.192	118122.	118370.	-248.	1675.	1.
0.196	0.196	119902.	120093.	-191.	1484.	1.
0.200	0.200	121683.	121683.	0.	1484.	1.
0.204	0.204	122801.	123101.	-300.	1184.	0.
0.208	0.208	123920.	124329.	-409.	775.	0.
0.212	0.212	125038.	125352.	-314.	461.	0.
0.217	0.217	126156.	126156.	0.	461.	0.


STATION RES
(O) INTERPOLATED BREACH HYDROGRAPH
(B) COMPUTED BREACH HYDROGRAPH
(•) POINTS AT NORMAL TIME INTERVAL
TIME (HRS)
0. 20000. 40000. 60000. 80000. 100000. 120000. 140000. 0. 0. 0. 0. 0.


STATION RES
INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)
20000.
40000.
60000.
80000.
100000.
120000.
140000.

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

				RATIOS APPLIED TO FLOWS
OPERATION	STATION	AREA	PLAN	RATIO 1 1.00
HYDROGRAPH AT	SA-RES	0.12	1	3032.
	(	0.30)	(	85.86)(
ROUTED TO	RES	0.12	1	126156.
	(	0.30)	(	3572.35)(
ROUTED TO	7+00	0.12	1	124668.
	(	0.30)	(	3530.21)(
ROUTED TO	50+00	0.12	1	109586.
	(	0.30)	(	3103.13)(
ROUTED TO	100+00	0.12	1	97891.
	(	0.30)	(	2771.96)(
ROUTED TO	140+00	0.12	1	93147.
	(	0.30)	(	2637.63)(
ROUTED TO	170+00	0.12	1	93009.
	(	0.30)	(	2633.72)(
ROUTED TO	200+00	0.12	1	92185.
	(	0.30)	(	2610.38)(
ROUTED TO	216+00	0.12	1	92334.
	(	0.30)	(	2614.59)(
ROUTED TO	224+00	0.12	1	92273.
	(	0.30)	(	2612.89)(
ROUTED TO	229+00	0.12	1	92169.
	(	0.30)	(	2609.92)(
ROUTED TO	237+00	0.12	1	91757.
	(	0.30)	(	2598.27)(

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1771.50	1768.00	1771.50
STORAGE	1861.	1591.	1861.
OUTFLOW	3032.	0.	3032.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	1771.50	0.00	1861.	127149.	0.00	0.23	0.00

PLAN 1 STATION 7+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	124668.	1730.2	0.22

PLAN 1 STATION 50+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	109586.	1547.4	0.25

PLAN 1 STATION 100+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	97891.	1446.9	0.28

PLAN 1 STATION 140+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	93147.	1310.0	0.32

PLAN 1 STATION 170+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	93009.	1141.1	0.32

PLAN 1 STATION 200+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	92185.	905.6	0.33

PLAN 1 STATION 215+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	92334.	823.8	0.33

PLAN 1 STATION 224+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	92273.	655.3	0.33

PLAN 1 STATION 229+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	92169.	605.0	0.33

PLAN 1 STATION 237+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	91757.	585.3	0.35

APPENDIX E

INFORMATION AS CONTAINED IN

THE NATIONAL INVENTORY OF DAMS

INFORMATION AS CONTAINED IN

THE NATIONAL INVENTORY OF DAMS

THIS SHEET TO BE FURNISHED BY

THE CORPS OF ENGINEERS

APPENDIX F

REFERENCES

REFERENCES

This is a general list of references pertinent to dam safety investigations. Not all references listed have necessarily been used in this specific report.

1. "Recommended Guidelines For Safety Inspection of Dams", Appendix D of ER 1110-2-106, Dept. of the Army, Office of the Chief of Engineers, Washington, D.C., 26 September 1979.

2. "HEC-1 Flood Hydrograph Package, Users Manual", The Hydrologic Engineering Center, U.S. Army Corps of Engineers, January 1973.

3. "Flood Hydrograph Package (HEC-1), Users Manual for Dam Safety Investigations", The Hydrologic Engineering Center, U.S. Army Corps of Engineers, September 1978.

4. HMR 33, "Seasonal Variations of Probable Maximum Precipitation, East of the 105th Meridian for Areas 10 to 1000 Square Miles and Durations from 6 to 48 Hours," U.S. Department of Commerce, NOAA, National Weather Service, 1956.

5. HMR 51, "All-Season Probable Maximum Precipitation, U.S. East of 105th Meridian for Areas from 1000 to 20,000 Square Miles and Durations from 6 to 72 Hours", U.S. Department of Commerce, NOAA, National Weather Service, 1974.

6. HYDRO-35, "Five-to-60 Minute Precipitation Frequency for the Eastern and Central United States", U.S. Department of Commerce, NOAA, National Weather Service, June 1977.

7. "Technical Paper No. 40, Rainfall Frequency Atlas of the United States", U.S. Department of Commerce, Weather Bureau, 1961.

8. Design of Small Dams, United States Department of the Interior, Bureau of Reclamation, Second Edition, 1973.

9. King, Horace W. and Brater, Ernest F., Handbook of Hydraulics, fifth edition, McGraw-Hill Book Co., Inc., New York, 1963.

10. "Flood Hydrograph Analyses and Computations", EM 1110-2-1405, U.S. Army Corps of Engineers, 31 August 1959.

11. "Technical Release No. 55, Urban Hydrology for Small Watersheds", U.S. Department of Agriculture, Soil Conservation Service (Engineering Division), January 1975.

12. "Hydraulic Design of Spillways", EM 1110-2-1603, U.S. Army Corps of Engineers, 31 March 1965.

13. "Standard Project Flood Determinations", EM 1110-2-1411, U.S. Army Corps of Engineers, 26 March 1952.

14. "Hydrologic and Hydraulic Assessment", Appendix D of EC 1110-2-188, U.S. Army Corps of Engineers, 30 December 1977.

15. "Reviews of Spillway Adequacy, National Program of Inspection of Non-Federal Dams", ETL 1110-2-234, U.S. Army Corps of Engineers, 10 May 1978.

16. Hammer, Mark J., Water and Waste-Water Technology, John Wiley & Sons, Inc., New York, 1975.

17. "Hydraulic Charts For the Selection of Highway Culverts", Hydraulic Engineering Circular No. 5, U.S. Department of Commerce, Bureau of Public Roads, December 1965.

18. 33 CFR Part 22, Final Rule, "Engineering and Design; National Program For Inspection of Non-Federal Dams", ER 1110-2-106, U.S. Army Corps of Engineers, March 24, 1980.

19. "Water Resources Data For New Hampshire and Vermont - Water Year 1977", USGS Water-Data Report NH-VT-77-1, U.S. Geological Survey, Boston, Ma., 1978.

20. "Climatological Data - May 1979 - New England", Volume 91, No. 5, National Oceanic and Atmospheric Administration, National Climatic Center, Asheville, North Carolina.

21. "Climatological Data - Annual Summary - New England", Volume 90, No. 13, National Oceanic and Atmospheric Administration, National Climatic Center, Asheville, North Carolina.

END

FILMED

8-85

DTIC